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OF THE
New England Water Works Association
1959-1960

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THE Association was organized in Boston, Mass., on June 21, 1882, with the object of providing its members with means of social intercourse and for the exchange of knowledge pertaining to the construction and management of water works. From an original membership of only TWENTY-SEVEN, its growth has prospered until now it includes the names of over 1000 men. Its membership is divided into six classes, viz.:

A Member shall be an officer or employee of a public or private water works, an engineer, chemist or other person qualified to aid or interested in the advancement of knowledge relative to water works.

An Honorary Member shall be a person of acknowledged eminence in some branch of water supply or of engineering.

A Junior shall be not less than eighteen years nor more than twenty-five years of age, a student or connected with water supply work.

An Associate shall be either a person, firm or corporation engaged in manufacturing or furnishing materials or supplies for the construction or maintenance of water works.

A Corporate Member shall be either a Water Board, Commission, Company or Municipal Corporation.

The initiation fees and annual dues are as follows:

	Initiation Fees		*Annual Dues*
Members	$ 3.00	Members	$ 6.00
Juniors	1.00	Juniors	3.00
Associates	10.00	Associate	25.00
Corporate Members	10.00	Corporate Members	15.00

This Association has at least eight regular meetings each year, of which five are held in Boston, one in northern New England, one in southern New England, and one, the annual convention, held in September or October on such date as the Executive Committee may designate.

That's Lock Joint Concrete Pipe!

Whether it be the largest concrete pressure pipe in the world, or a small diameter drainage pipe;
. . . . pipe for the highest heads common to water works practice, or pipe for only nominal pressure or partial vacuum;
. . . . pipe to resist extremes of overburden, or pipe to meet the complexities of industrial or plant piping;
. . . . LOCK JOINT can provide all these, and more.

You will also find it a dependable, durable, maintenance-free pipe of sustained high carrying capacity which is backed by a company with over a half century's experience in the development and production of concrete pressure, sewer, culvert and subaqueous pipe.

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East Orange, New Jersey

Sales Offices: Chicago, Ill. • Columbia, S. C. • Denver, Col. • Detroit, Mich.
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Pressure • Water • Sewer • REINFORCED CONCRETE PIPE • Culvert • Subaqueous

PIPE THAT DOESN'T GET COLD FEET IN WET TRENCHES

This cast iron pipe was installed to carry water to an important industrial site in Indiana.

At the time of installation, weather conditions were severe; the trenches were wet and muddy.

Still, the cast iron pipe was easily and speedily assembled, enabling the contractor to complete the job well ahead of the deadline.

Cast iron pipe does not get cold feet over rugged soil conditions. Its rugged design, corrosion-resistant qualities, and bottle-tight joints will withstand the most severe conditions.

Modern, cement-lined cast iron pipe will deliver a continued full flow of water for at least a century . . . and rarely requires repairs or replacement.

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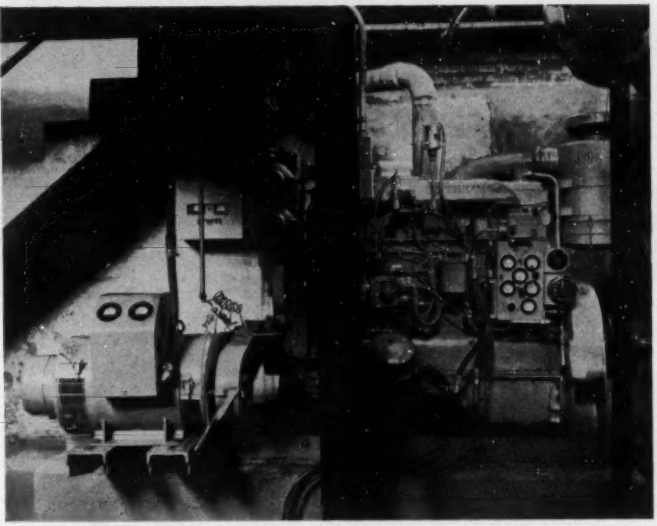
CAST IRON PIPE

THE MARK OF THE 100-YEAR PIPE



CUMMINS DIESEL PUMPING ENGINE AND GENERATOR SET
HAGGETT'S POND PUMPING STATION
TOWN OF ANDOVER, MASSACHUSETTS

A Cummins Model NHRS heat-exchanger-cooled engine provides power for a 2500 GPM water pump and, in addition, drives a 10 KW generator with packaged controls to give station lighting and power miscellaneous small motors.

Camp, Dresser and McKee of Boston, Massachusetts, were engineers for this unusual application of diesel power.

Installation was completed by R. H. White Construction Co., Inc., of Auburn, Massachusetts.

CUMMINS DIESEL OF NEW ENGLAND, INC.

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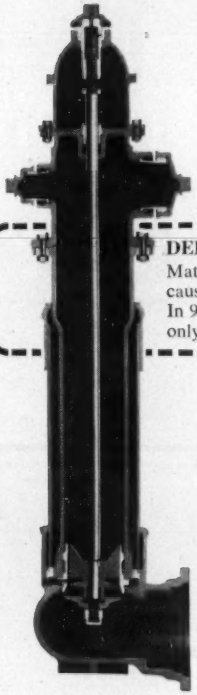
ONLY MATHEWS HAS ALL THE DESIRABLE FEATURES



What do you look for in a hydrant? Dependability? . . . replaceable barrel? . . . conventional packing? . . . "O" ring seal? . . . stop nut to prevent stem from buckling? We could double this list and still not cover all the features we build into Mathews Hydrants *by design*. Mathews has them all.

DEPENDABILITY

Mathews Hydrants are always ready for service, because their design is fundamentally sound and simple. In 90 years we have never changed the basic design—only refined it, by detail.

AND ALL THESE OTHER FEATURES, TOO:

All working parts contained in replaceable barrel • Head can be faced in any direction • Replaceable head • Any nozzles you specify • Nozzle levels raised or lowered without excavating • Leakproof because stuffing box is cast integral with nozzle section • Operating thread only part requiring lubrication • Protection case of "Sand-Spun" cast iron for strength, toughness, elasticity • Available with bell, mechanical joint or flange pipe connections

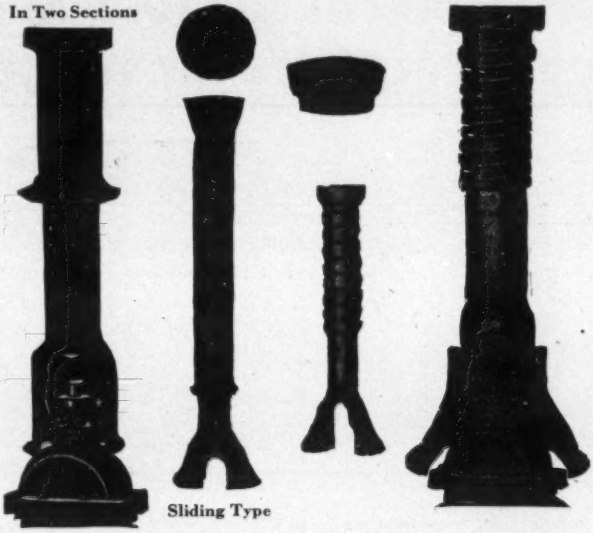
Conform to latest AWWA specifications

R. D. WOOD COMPANY

Public Ledger Building, Independence Square, Philadelphia 5, Pa.

Manufacturers also of Mathews Flange Barrel Hydrants, R. D. Wood Hydrants, R. D. Wood Gate Valves, and "Sand-Spun" Pipe (centrifugally cast in sand molds)

In Two Sections



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1. *Standardization*
2. *Dependability*
3. *Quick Delivery*

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FIT 24", 30", AND 36"

METER PIT BARRELS

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Ford Catalog No. 56 contains full information. Send for your copy today. FREE.

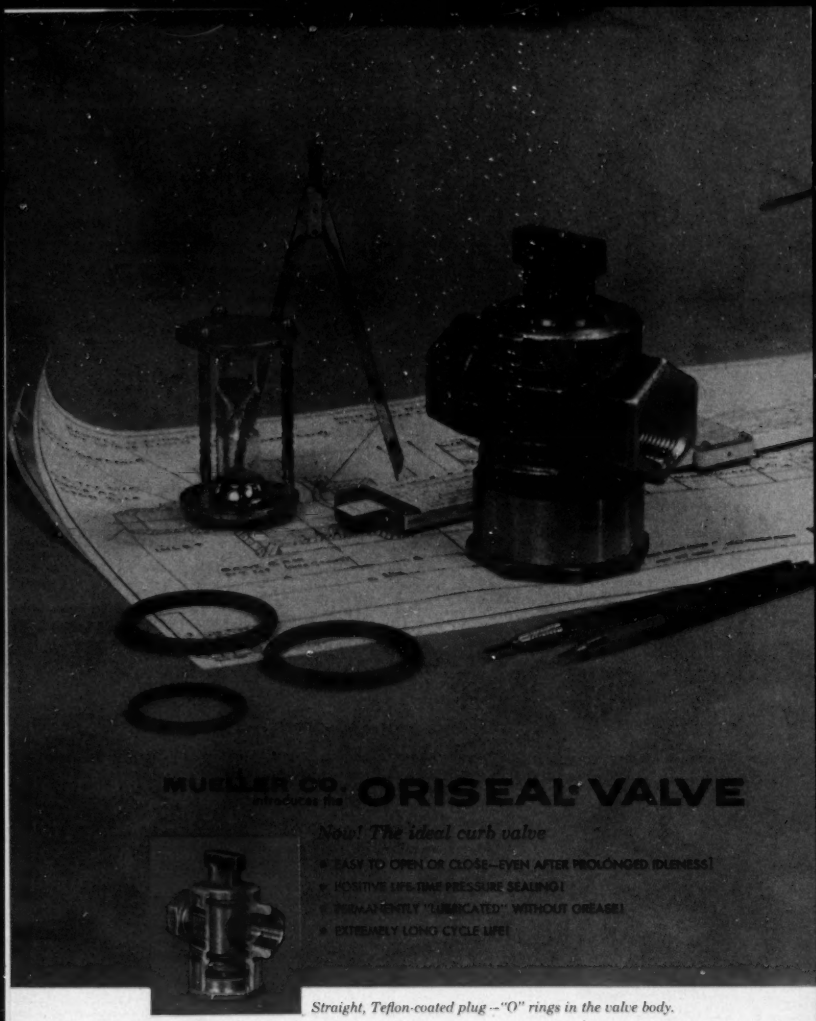
For covering large meter pits the Monitor Cover is most useful. With lid opening large enough to permit meter reader to enter setting, this cover is used over pits containing 1½" or 2" meters, or multiple settings of smaller meters. An inner lid is available if extra frost protection is desired.

The cover is equipped with the Ford Lifter Worm Lock that opens lid easily, even when frozen.

FOR BETTER WATER SERVICES FORD

Manufactured by THE FORD METER BOX COMPANY, INC.

Wabash, Indiana


MUELLER CO. introduces the ORISEAL VALVE
Now! The ideal curb valve
• EASY TO OPEN OR CLOSE—EVEN AFTER PROLONGED IDLENESS!
• POSITIVE LIFE-TIME PRESSURE SEALING!
• PERMANENTLY "LUBRICATED" WITHOUT GREASE!
• EXTREMELY LONG CYCLE LIFE!

Straight, Teflon-coated plug—"O" rings in the valve body.

Write today for complete information or contact your Mueller representative.

MUELLER CO.,
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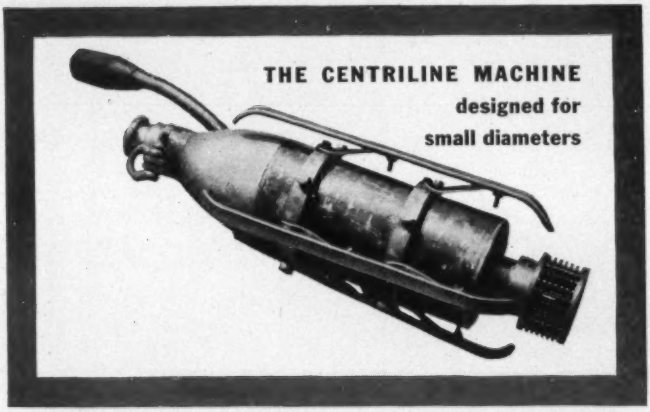
Factories at: Decatur, Chattanooga, Los Angeles; In Canada: Mueller, Limited, Sarnia, Ontario.

Vol.
19

NOW! THE CENTRILINE PROCESS is available for 6" to 14" mains, too!



Yes, your city's small but important transmission and distribution lines can also regain their original flow capacity and pressure through the universally accepted Centriline Process of centrifugally applying a cement-mortar coating to the pipe walls. If some of your lines inadequately serve your customers' requirements, including higher demands for fire protection, investigate the numerous advantages of Centrilining your mains now.

The new, small diameter Centriline Machine eliminates most excavations at valves, laterals and corporation cocks. By eliminating these fixed costs, lengthening the distances between access openings and permitting faster lining speeds, the new Centriline Machine has really reduced the cost of lining small mains. So much so, in fact, that every water works operator should reevaluate the economics of small pipe rehabilitation.

Send today for your copy of our illustrated booklet which fully describes how Centriline can help you salvage worn out pipes from 6" to 144" in diameter permanently and for much less than you would imagine.

CENTRILINE CORPORATION

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Branch Offices in Principal Cities of the United States, Canada and Latin America.

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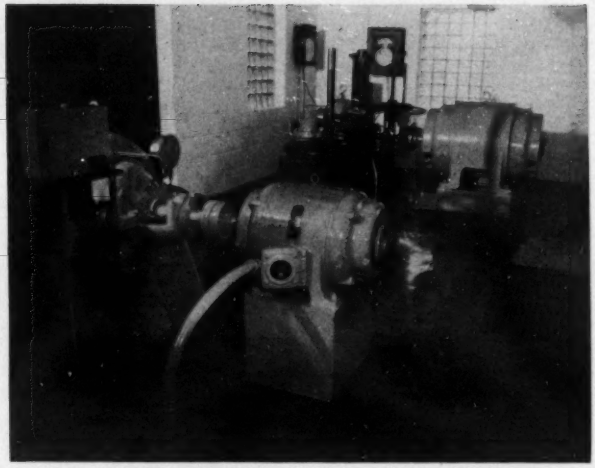

Marblehead Booster Station
Marblehead, Mass.

Shown in foreground is 15 HP motor driven Fairbanks-Morse 12″ split case centrifugal pump — 3500 GPM at 9′ TDH. Other unit is 100 HP motor driven Fairbanks-Morse 14″ split case centrifugal pump — 5150 GPM at 65′ TDH.

HALEY & WARD, ***Consulting Engineers***
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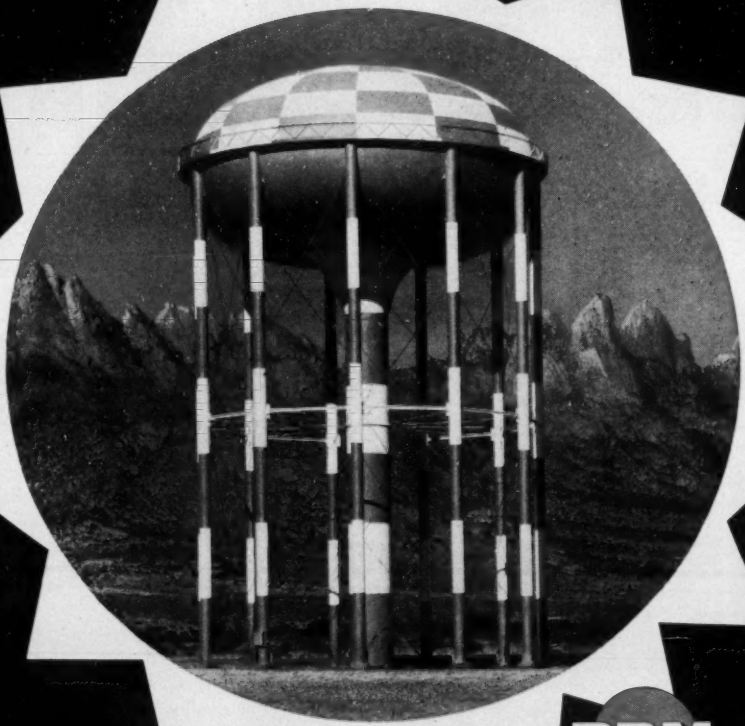
Telephone: LAfayette 3-3600

SOMETHING NEW UNDER THE SUN at WHITE SANDS, New Mexico



PDM
ELEVATED STEEL TANKS

THIS TORO-SPHERICAL TANK BY Pittsburgh-Des Moines

New design—new economy in purchase for the larger storage capacities. The PDM TORO-SPHERICAL Elevated Tank requires less steel in construction, yet has the same strength and meets the same design criteria as conventional low head range tanks. Pictured is the prototype at White Sands, N.M. Capacity one million gallons, height 99 ft to bottom, head range 35 ft. Write for details.

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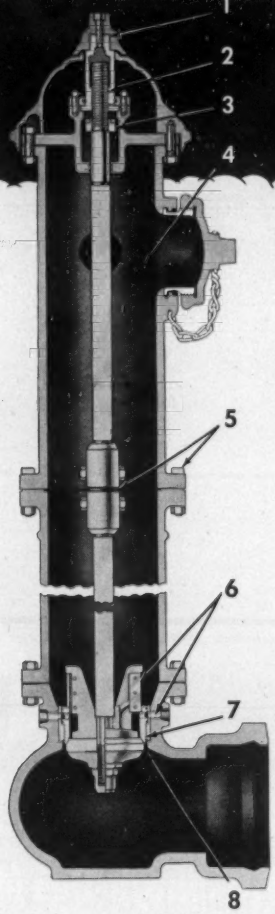


AS ADVANCED AS TOMORROW...

NEW!

SMITH MODEL H100 HYDRANT

proudly joins the time-proven Smith product line. No other hydrant has ALL these features:

1. Weatherproof — Tamperproof Operating Nut.
2. Permanently Lubricated Operating Mechanism.
3. Top Travel Stop Nut Prevents Rod Bending.
4. Streamlined Openings Assure Maximum Delivery.
5. Frangible Bolts and Coupling Prevent Standpipe Damage.
6. Multiport Drain Mechanism Operates Automatically.
7. Bronze-to-Bronze Valve Assembly — Self-Sealed "O" Rings.
8. Compression Type Valve Prevents Flooding.

Write for details.

THE A.P. SMITH MFG CO.

HYDRANTS — VALVES — SINCE 1896

EAST ORANGE NEW JERSEY

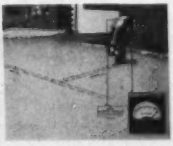
M-Scope "Master" Electronic Witch
Leak Survey Instrument and Pipe Locator

The transistorized "MASTER" Leak Detector—Pipe Finder Combination is the first dual purpose instrument with separate and independent leak detector and pipe finder circuits. Now you can have two instruments in one without any compromise in performance. The "MASTER" Leak Detector—Pipe Finder incorporates the very latest advances in electronic construction and circuitry in addition to having an exceptionally long battery life. An ideal instrument for any Water Department.

Finding a leak under pavement

Locating a leak at the valve

Location of dead ends



Locating a service

Write for information, demonstration or trial.

PIPE LINE EQUIPMENT

JOSEPH G. POLLARD CO., INC.

PIPE LINE EQUIPMENT

PIPE LINE EQUIPMENT

Place your next order with POLLARD

If it's from POLLARD . . . It's the Best in Pipe Line Equipment

NEW HYDE PARK • NEW YORK

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Why this "magnetic" meter is always easy to read

In Rockwell Sealed Register meters a powerful magnetic coupling transmits every motion of the measuring chamber directly to a hermetically sealed register.

This construction permits the sealed register to be positioned high and dry above the flowing stream. Condensation and dirt, the bane of the meter reader, just can't occur. In fact, this meter will run under water without harm.

The sealed register assembly contains all the gearing both for register operation and for intermediate reduction. Gears and pinions are of the free running instrument type. They offer only token resistance to the measuring element and so improve the accuracy and response of the meter especially at low flows.

All this, plus a simplified design in which only two parts operate in water means less wear, less chance for corrosive attack and of course easier maintenance at lower cost. Get facts now. Write to Rockwell Manufacturing Company, Pittsburgh 8, Pa.

POWERFUL MAGNETIC DRIVE

Illustrating how the driver magnet (held in hand) revolves around a watertight compartment at base of register to push the follower magnet, which in turn drives the register shaft.

SEALED REGISTER METERS

another fine product by

ROCKWELL

Warren Foundry & Pipe Division

of

SHAHMOON INDUSTRIES, INC.

Manufacturers of

CAST IRON PIPE & FITTINGS

Made in Accordance with Latest Specifications

Including TYTON Joints

Either Cement Lined or Tar Coated

Pipe Made in Everett — Centrifugally Cast in Sand Lined Molds in 16′ L/L

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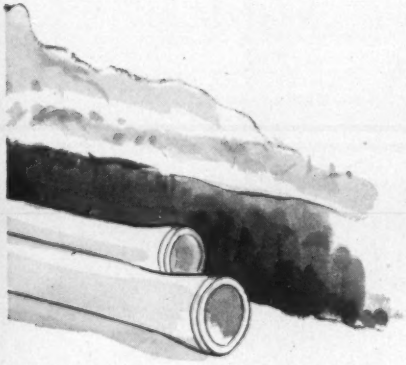
U.S. cast iron **PIPE** FOR WATER, SEWERAGE AND

saving's in style again!

Today . . . figuring costs . . . you need a sharper-than-ever pencil. And the closer and sharper you figure, the more you need Tyton.®
The facts: Tyton Joint® pipe is easy to assemble . . . can be laid faster even by green crews.
Tyton cuts overhead, bookkeeping, storage costs—only one accessory needed. It increases working days . . . can even be laid in rain or wet trench.
Simple, speedy, sure, Tyton Joint pipe is tailormade to meet today's rising costs. Judge for yourself. For the full story, write or call.

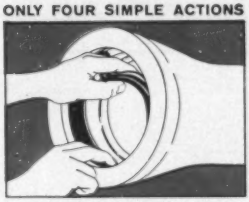
U.S. PIPE AND FOUNDRY COMPANY
General Office: Birmingham 2, Alabama
A Wholly Integrated Producer from Mines and Blast Furnaces to Finished Pipe



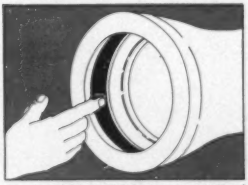
INDUSTRIAL SERVICE CAST IRON

TYTON®

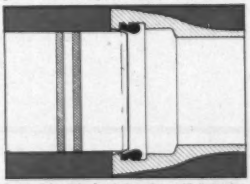
ONLY FOUR SIMPLE ACTIONS



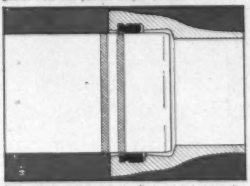
Insert gasket with groove over bead in gasket seat . . . a simple hand operation.



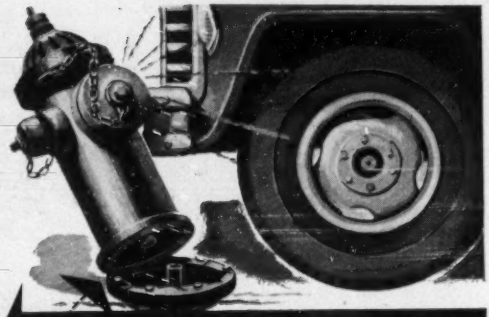
Wipe film of Tyton Joint® lubricant over inside of gasket. Your receiving pipe is ready.



Insert plain end of entering pipe until it touches gasket. Note two painted stripes on end.



Push entering pipe until the first painted stripe disappears and the second stripe is approximately flush with bell face. The joint is sealed . . . bottle-tight, permanently! The job's done . . . . fast, efficiently, economically. Could anything be simpler?



THIS DEMONSTRATION

proved the advantages of

THE NEW EDDY BREAK-FLANGE HYDRANT

• Clean breaking action of the new EDDY break-flange hydrant is clearly demonstrated in this film strip of a test made recently for Mr. Herbert Campbell, Town Engineer, Wayne, New Jersey. Just as it was designed to do, the hydrant broke off at the break-flange and the coupling sheared at the weak point.

Mr. Campbell was so impressed that he changed the specifications on the spot... wrote new specifications around the EDDY Hydrant. Another EDDY feature which appeals to economy-minded waterworks officials is that these hydrants, with the break-flange feature, are the same as standard EDDY models in all other respects. Working parts are interchangeable and no separate inventory of repair parts is required.

We'll be glad to help you arrange a similar demonstration. It pays to standardize on EDDY hydrants and valves.

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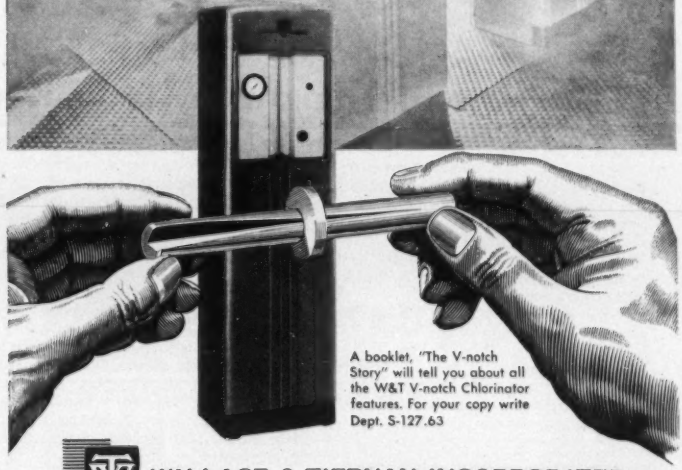
the V-notch
is the difference in chlorine control

Safe chlorine control is the job of a chlorinator.
This control, the V-notch, does the job completely.
Yet it appears to be simply a grooved plug in a ring.
It is, of course, more than that... much more.
The groove in the plug is precision milled and shaped so that when you turn the control knob on your chlorinator and the plug slides through the ring, the size of the opening between the groove and the ring changes an exact amount. You meter chlorine exactly and simply because every position of the plug in the ring makes a repeatable orifice size—a repeatable chlorine flow rate.
Only in a W&T V-notch Chlorinator is control so simple, so true, so precisely right.
And, of course, the right plastics make the whole chlorinator chlorine-proof.



A booklet, "The V-notch Story" will tell you about all the W&T V-notch Chlorinator features. For your copy write Dept. S-127.63

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COMPARED TO THE MONEY IT EARNS

Figure it this way:

Write down the revenue you earn each year from the average meter: $_____. Now multiply this by seven (the number of years many Commissions permit a meter to stay in service before retest) or by the number of years required by *your* local regulations.

What have you got? $_____. A figure somewhere in the neighborhood of $350 to $700? That's the money you entrust to just one water meter while your back is turned. That's the money it earns, twelve to twenty times its price.

Like any instrument, a meter runs slow as it wears, and gives water away before it is brought in for test and repair. Now if *one* meter, by virtue of better design and workmanship, stays accurate and brings in just 1% more revenue over the years, how many extra dollars is that worth to you? $3.50? $7.00?

That's how much more you could afford to pay for a Trident water meter.

There's extra value, too, in Trident's interchangeable design. New Trident parts fit old Trident meters. This "no obsolescence" feature insures automatic modernization, keeps repair costs down and protects your meter investment.

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METERS AND INSTRUMENTATION — Complete Line of Primary Metering Elements: Venturi Tubes and Nozzles; Dall Flow Tubes; Telemeters for Flow, Pressure, Temperature, Level, Position and Electrical Quantities; Velocity-type Meters.

FEEDERS AND WEIGHERS — Wide Range of Volumetric and Belt Gravimetric Feeders for Dry Materials and Liquids; Belt Weighers; Chlorine Gas Feeders.

VALVES — Butterfly Valves and Operators for Low and High Pressure Applications.

SYSTEMS — Supervisory Control Systems; Filters & Filter Control Systems; Volumetric and Gravimetric Systems for Automatic Blending, Feeding, Treating, Sampling.

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BUILDERS-PROVIDENCE • PROPORTIONEERS • OMEGA

METERS • FEEDERS • CONTROLS / CONTINUOUS PROCESS ENGINEERING

Single Responsibility for a system means time savings in coordination; compatible components, peak performance, maximum reliability and minimum initial and long term costs. **B-I-F INDUSTRIES, INC.**, 366 **Harris Ave., Providence 1, R. I.**

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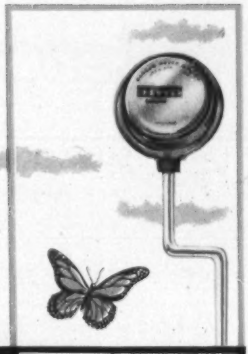

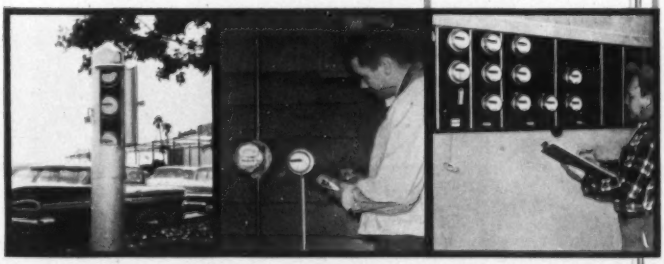

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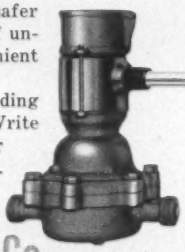

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DEVELOPMENT OF GROUND-WATER SUPPLIES FOR THE BRUNSWICK AND TOPSHAM WATER DISTRICT

BY E. BOYD LIVESAY*

[*Read September 23, 1959.*]

HISTORICAL

The Towns of Brunswick and Topsham in Maine have for years faced a dilemma certainly not unique in either New England or the United States, but none the less most lamentable. While the two towns have for years had a history of water shortage, between them lies a river, the Androscoggin, with a daily mean flow of 4200 cfs of water to the sea, so grossly polluted as to be impracticable to utilize for water-supply purposes. The result has been that during a period of 55 years the Towns of Brunswick and Topsham have spent many thousands of dollars searching for a ground-water supply, when even today 1/2000 part of the water flowing by would satisfy their needs.

The present Water District was formed in 1903, and took over holdings of the Maine Water Company, which served the towns with water obtained from the river. By that time the Androscoggin River had become so polluted that citizens of the towns complained bitterly about the quality of the water as well as the dangers of consuming it, and finally forced the formation of the Water District. The newly formed District immediately started looking for a new source of supply, and hired Mr. Freeman C. Coffin, Engineer, of Boston, Massachusetts, to supervise the exploratory work for a supply and to design the supply facilities. After examination of the area, which has no large surface sources other than the Androscoggin River available, Mr. Coffin advised a search for a ground-water supply.

This search culminated in the location of the Jordan Avenue

*Superintendent, Brunswick and Topsham Water District, Brunswick, Me.

Well Field which served the District continuously from 1906 to 1955 as its only real source of supply (See Fig. 3 for location). This area was test pumped in 1904 from March 28th to April 11th at the average rate of 500,000 gallons daily with a maximum drawdown of 11.4 feet. As a result of this testing and a study of the watershed area, Mr. Coffin estimated the safe yield of this area to be one mgd (million gallons per day). He recommended development of this site of approximately 120 acres by the installation of fifty 2½″ driven wells connected to a pumping station by a suction header varying in size from 6 to 16 inches. The pumping equipment consisted of one 1400 gpm and one 1000 gpm steam pump, along with appurtenant vacuum and auxiliary pumps.

This well field and pumping station system was built by B. F. Smith & Bros., and put into operation on January 12, 1906, but even before that summer was over, it was discovered that additional wells would be required and accordingly 33 more 2½″ driven wells were added. In 1923 the District suffered a severe water shortage as a result of the plugging of a large number of 2½″ well screens. At that time 40 of these wells were completely renovated. Later, in 1929, three 40′ diameter by 35′ deep dug wells were added to further augment the supply.

Adequacy of Supply

During the years from 1924 to 1952 individual 2½″ wells, and suction header branches containing several wells, were dropped from the system when it was found that the water from these wells had exceptionally high iron or hydrogen sulfide content. This source, which we refer to as the Jordan Avenue Well Field, had been reduced to sixty 2½″ wells and the three dug wells in 1952 when Brunswick and Topsham again experienced a severe water shortage, due to extreme drought, and a large increase in population. In fact in July of 1952 the water level in the Topsham standpipe had been drawn down to 5′ as compared to a normal level of 43′.

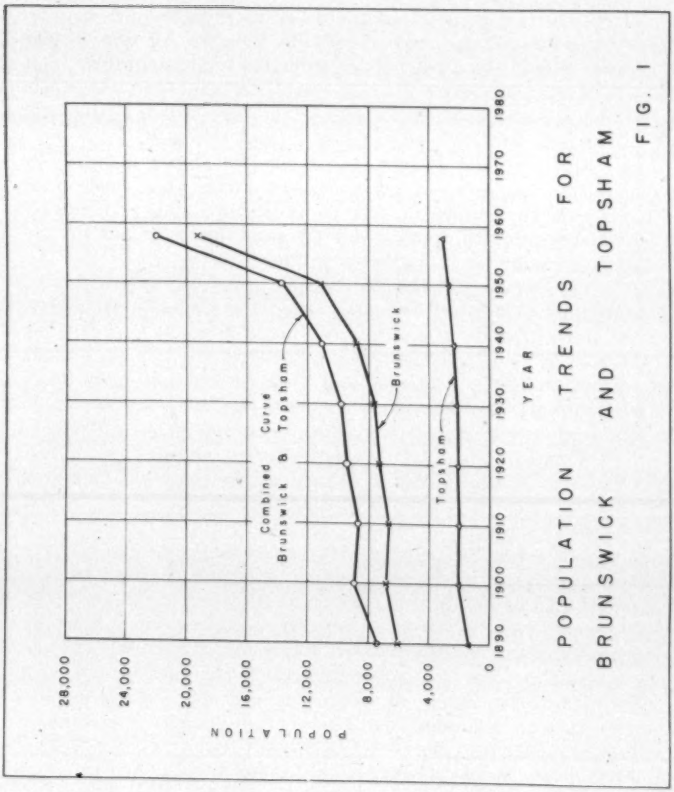
A rather drastic population increase was brought about by the re-activation of the Brunswick Naval Air Station in 1951. This station was established early in World War II, but was deactivated in 1946. In 1950 the U. S. Census figures showed

Brunswick and Topsham to have a combined population of 13,619. By 1953 this figure had jumped to 18,899, an increase of nearly 40%. During the same period the average daily demand of the District increased from 601,000 gpd (gallons per day) in 1950 to 830,000 gpd in 1953. At this time the Air Station had grown to only about one half its estimated total complement, and was itself using about 250,000 gpd. They estimated that their station demand would increase to 450,000 gpd within a year, and at one time said their demand would reach two mgd.

Figures 1 and 2 show graphically the abrupt increases in population and water consumption which have taken place since 1950. The effect of World War II Naval Air Station activities on water consumption accounts for the pronounced "hump" in the water consumption trend shown on Fig. 2.

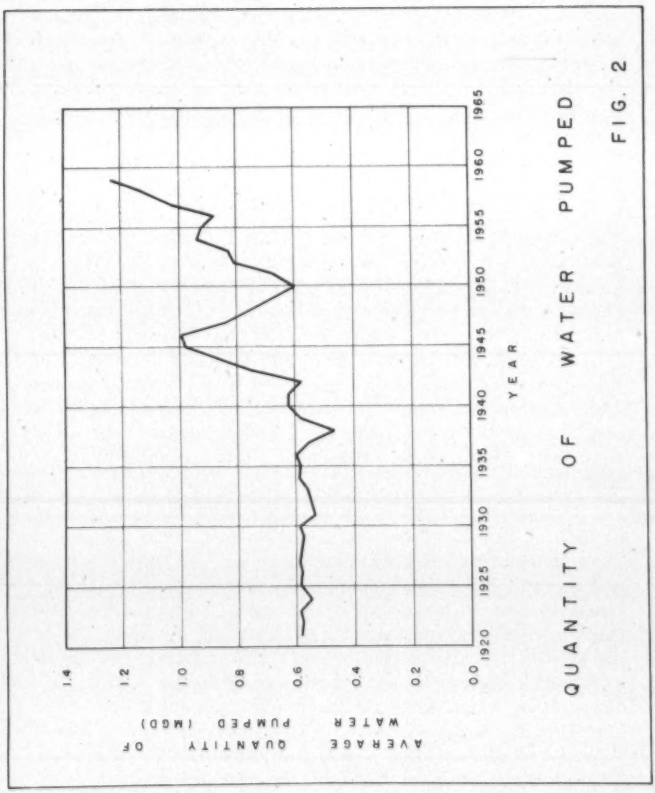
The critical situation of drought, water shortage and ever increasing population obviously called for a greatly intensified search for ground-water supply. We might interpose here that the District has conducted virtually a continuous search for ground water since 1903. Brunswick and Topsham are both quite liberally studded with exploratory test wells put in by our own crew, one of our local well drillers, Mr. Wilfred E. Jones, the Lane-New York Co., Inc., D. L. Maher Company, and R. E. Chapman Company. In 1951 and 1952 in particular, a concerted effort was made by all of the above-mentioned concerns as well as by the Weston Observatory of Weston, Massachusetts, which conducted a seismic survey of the area. Altogether, sixty exploratory wells were placed and nine seismic surveys conducted in this two-year period. Since 1903 there has been a total of 191 exploratory wells of record placed in this area; however, up to the end of 1952 the main use of this exploratory work was to tell us where not to look.

At this time the Brunswick Naval Air Station was also searching for a supply of their own, and were considering developing a surface supply and filter plant to produce 2 mgd. After several conferences between the District and Brunswick Naval Air Station officials, it was finally decided to make a joint effort of this endeavor. Advice of Metcalf & Eddy, Consulting Engineers of Boston, was sought, and it was decided to proceed immediately to improve the yield of the Jordan Avenue Well Field.



POPULATION TRENDS FOR BRUNSWICK AND TOPSHAM

FIG. 1



QUANTITY OF WATER PUMPED

FIG. 2

Improvements to Jordan Avenue Well Field

Metcalf & Eddy recommended the addition of a new well field at the Jordan Avenue site, consisting of 76 new 2½" driven wells, together with a new pumping station to pump from both well fields. The new pumping station was to be constructed above the maximum flood level, because the existing station had been flooded out and made inoperative on previous occasions during major floods.

The 76 new wells were installed by the D. L. Maher Company of Woburn, Massachusetts, during the winter of 1952. Bids were received for the construction of the new pumping station and suction main system in June of 1953. Unfortunately, the bid price for this construction was $58,000 more than the $153,000 estimated. It was decided to complete the new well field, but to eliminate the new pumping station because of the high cost. This decision was influenced by success in locating promising new sites for well supplies at Durham Road and Williams Farm in early 1953 (discussed hereinafter). Subsequently, the existing Jordan Avenue pumping station was completely renovated and facilities provided for the quick removal of electric motors and other equipment susceptible to water damage in the event of a major flood.

During the installation of the new suction mains in August of 1953, it was discovered that the effective diameter of the old 12-in suction main had been reduced to about 7½" resulting from rust deposits. Further examination also revealed that a large percentage of the old well points were clogged. A decision was then made to completely renovate the old well field by cement lining the existing suction headers in place and installing new wells. This work required that the Jordan Avenue supply be taken out of operation, which was made possible by pumping a well at the new Durham Road site directly into the distribution system through a temporary force main laid on the surface of the ground. Testing of the Durham Road site is described in more detail hereinafter.

The results of the improvements to the Jordan Avenue well field were shown dramatically when the new system was put into operation. The pumping rate was increased from an average of

600 gpm (gallons per minute) in January and February of 1953, to an average of 1000 gpm during the same months of 1954, using exactly the same pumping equipment.* Moreover, the quality of the water from this source has also shown a definite improvement, as evidenced by lower iron and color content. The safe yield of the Jordan Avenue Well Field is now estimated at approximately 1.5 mgd.

Exploratory Work for New Supplies

Early in 1953 the District signed a "no water—no pay" contract with the R. E. Chapman Company of Oakdale, Massachusetts, to do additional exploratory work for a ground-water supply. This exploratory work consisted principally of "jetting" a great many 2″ wells in numerous areas within two miles of our existing distribution system. This method, which consists of hydraulically jacking a 2″ well pipe into the ground, meanwhile forcing a water jet through it at pressures up to 300 psi, is an extremely rapid way of exploring for water-bearing gravel.

This exploratory work led to the Williams Farm-Durham Road-Taylor Farm sites about 2½ miles northwest of the Brunswick Town Hall (See Fig. 3 for location). These three sites lie in a long, buried valley which trends in a southwest-northeast direction for at least two miles. The valley extends south of Durham Road and passes under the Androscoggin River to the northerly side of the river.

This old valley is cut into hard bedrock and was partly filled during the glacial period with discontinuous deposits of clay, silt, sand and gravel. Some of these valley-fill deposits consist of highly permeable sand and gravel capable of yielding substantial amounts of ground water to well-constructed wells. Judging from bedrock outcrop, the valley is about one half mile wide. Test well results show that the depth to bedrock ranges from 100 to 160 feet below the existing ground surface.

Exploratory work conducted at each of the above sites by the installation of 2″ jetted test wells and 2½″ driven test wells revealed water-bearing sand and gravel deposits at considerable depths below the ground surface which yielded water freely when pumped. Large-diameter test wells were installed at each site and

* "The Case for a Positive Well-Field Maintenance Program," E. B. Livesay, Sept., 1956, NEWWA Journal.

continuous pumping tests conducted to determine the yield and to provide information on water quality. The results of these continuous pumping tests, and the permanent works which followed, are described below.

Durham Road Supply

The location of the Durham Road gravel-packed well is shown on Fig. 3. This well was installed early in 1953 as part of the R. E. Chapman "no water—no pay" contract. The well is 101 feet deep and contains 20 feet of 12-inch Everdur well screen at the bottom.

A pumping test of 17 days' duration was conducted by the R. E. Chapman Company in June of 1953. The test was conducted at the rate of 1000 gpm for the first eight days of the test, but was reduced to 720 gpm for the last nine days of the test. The maximum drawdown in the gravel-packed well was approximately 35 feet at a pumping rate of 720 gpm, representing a specific capacity of 20.6 gpm per foot of drawdown. The water level in four observation wells continued to drop a few inches per day throughout the pumping period. When pumping ceased, recovery in the various observation wells was very slow. Six weeks after completion of the pumping test, the water level in some of the observation wells had recovered only to within 4 feet of the original static level.

It was then decided to run a 3 months' pumping test on the Durham Road well, pumping the water directly into the distribution system. We might add here, with some emphasis, that fear of another severe water shortage as in 1952 was a consideration in reaching this decision. The Navy provided 7500 feet of 8-in quick-coupling pipe which was laid on the surface of the ground and connected to the existing distribution system. A deep-well turbine pump and diesel engine were obtained and the system was put into operation on August 7, 1953, the pump being run continuously until October 26, 1953, a total of 80 days. The pumping rate averaged about 550 gpm during most of the period, but fell off to about 500 gpm during the final week of the test. As has been mentioned previously, the use of this temporary supply enabled the District to take the Jordan Avenue supply out of operation for the purpose of renovating the old well field.

During the last day of pumping, at the rate of 540 gpm, the drawdown in the gravel-packed well was 35 feet, representing a specific capacity of 15.4 gpm per foot of drawdown. The water level in the four observation wells continued to drop throughout

TOPSHAM
TAYLOR FARM GRAVEL-PACKED WELLS NO. 3 & 4
HOLDEN FARM FUTURE WELL SUPPLY
WILLIAMS FARM GRAVEL-PACKED WELL NO. 2
DURHAM ROAD GRAVEL-PACKED WELL NO. 1 (NOT IN USE)
JORDAN AVENUE WELL FIELD
BRUNSWICK AND TOPSHAM WATER DISTRICT
LOCATION MAP - SOURCES OF SUPPLY
SCALE - Miles
1.0 0.5 0 1.0

FIG. 3

the test averaging about one inch per day during the last few weeks of pumping. When pumping ceased, recoveries were extremely slow (see above).

No immediate decision was made relative to development of the Durham Road supply because it was evident that development of the Williams Farm well described below was the logical

first step in increasing the District's supply. The consulting firm of Camp, Dresser & McKee was retained in 1954 to design and supervise construction of the Williams Farm supply and to review the whole ground-water picture. In their subsequent report on the Durham Road-Williams Farm-Taylor Farm supplies, they stated the following relative to the Durham Road supply:

> "The Durham Road well has not been pumped since October 26, 1953. In October of 1954, one year after the end of the pumping test, the water level in the observation wells had failed to recover to the original static level by about 4 ft and a period of almost 2 years elapsed after pumping before recovery was complete.
>
> "In view of the above, it is evident that the safe yield of this source is very substantially less than the 500 gpm pumping rate. Continuous pumping over a very long period of time would be required to make a reliable estimate of the safe yield of this source. The drainage area at this source, measured from the U. S. Geological Survey topographical maps, is approximately 0.2 sq mi and the well is located within the same drainage area as the Williams Farm well, which is about one mile downstream. Pumping from the Durham Road well would, in our opinion, affect the yield of the Williams Farm well. In view of the relatively limited yield of this source, together with the high cost of connecting to the existing system because of rock excavation, we recommend against consideration of this source for additional supply in the near future. We do recommend, however, that this source be retained and protected for possible future use at such time as it may be desirable to extend the distribution system to provide service to this section of the town."

Williams Farm Supply

The location of the Williams Farm gravel-packed well is shown on Fig. 3. This well was also installed early in 1953 as part of the R. E. Chapman Company "no water—no pay" contract. The well is 78 feet deep and contains 15 feet of 12-in Everdur screen at the bottom. The drainage area at this well is approximately 1.5 square miles.

A 5-day pumping test was conducted by the R. E. Chapman Company during the period September 22 to September 27, 1953, at a pumping rate of 520 gpm. The maximum drawdown was 25.5 ft, representing a specific capacity of 20.4 gpm per foot of drawdown. The maximum drawdown in an observation well 6½ ft

north of the gravel-packed well was 19 ft, while the maximum drawdown in an observation well 150 ft southwest was 4.4 ft. The water level in the gravel-packed well continued to drop throughout the 5 days of pumping at a rate of 15 inches per day. No recovery readings were taken.

An additional pumping test was conducted from June 23 to June 26, 1954, a period of three days. The pumping rate was 520 gpm and drawdown characteristics were similar to those observed during the previous 5-day test described above. Two additional observation wells were installed for the 3-day test and many water-level observations were made on the main well and observation wells (See Fig. 4 for location of wells), A logarithmic plot of drawdown vs. time was prepared from this data which, when projected, indicated that the well could be pumped at a rate of 700 gpm for a period of 40 years without lowering the water level below the top of the screen. While this method of estimating the safe yield of ground-water supplies has had some recent popularity, the fallacy of applying the method to small drainage areas such as the Williams Farm site was clearly shown by subsequent long-term pumping tests on this supply.

In August of 1954 the District laid a temporary 8-in pipeline on the surface of the ground from the Williams Farm Well along River Road to the existing distribution system near the standpipe, a distance of about 1¾ miles. The well was pumped continuously into the distribution system from August 10 to September 11, 1954, a total of 31 days, at a pumping rate of approximately 550 gpm. The maximum drawdown in the gravel-packed well was 32 ft, representing a specific capacity of 17.2 gpm per foot of drawdown. The water levels in all wells continued to drop more or less uniformly at the rate of several inches per day throughout the period of the test. The test was terminated on September 11, 1954, as a result of Hurricane "Edna," when the temporary diesel pumping installation was submerged by the flood accompanying the hurricane. In spite of the excess of water at the termination of the pumping test, a period of about two months elapsed before the wells recovered to their original level.

Estimates of the safe yield of the Williams Farm well as presented to the District ranged from 500 to 700 gpm. When Camp, Dresser & McKee was retained to prepare plans and super-

vise construction of the permanent works, they stated that they were "reluctant to accept these estimates, as our own study of the situation indicates that the safe yield may be substantially less than these figures. We nevertheless recommend a pumping capacity of 700 gpm for the Williams Farm well on the basis that the well may be pumped continuously at the full 700 gpm for considerable periods of time to meet heavy summer demands, even though the continuous yield during extended drought periods may be substantially less."

The location of the permanent Williams Farm works is shown on Fig. 5. The pumping station consists of a brick superstructure 14 feet by 21 feet in plan dimensions, on a concrete substructure. The station contains a 700-gpm deep well turbine pump driven by a 75 HP electric motor, connected by a right-angle gear drive to a standby diesel engine. The pump may be operated automatically by pressure controls, time clock or telemetering system. Fluoridation equipment is provided in the basement. Water is discharged from the well through 1600 ft of 12-in cast iron mains to River Road, and thence through about 7200 ft of 16-in cast iron main to the existing standpipe. The 16-in main was designed with capacity for full development of the Taylor Farm supply (described below) which was anticipated to follow the Williams Farm construction.

The new Williams Farm pumping station and force main was put into operation on February 15, 1955, and pumped more or less continuously at an average rate of about 700 gpm until May 23, 1955. The water level dropped continually in all the observation wells during this entire period. At the end of this period, the drawdown in the gravel-packed well was 58 ft, equivalent to 12 gpm per foot of drawdown, and the water level had been lowered to 5 ft above the top of the screen. At this point the water level in all of the wells was dropping uniformly 2 inches to 3 inches per day. After May 23, 1955, the pumping rate was maintained at 700 gpm but the hours of pumping per day were reduced to produce 0.4 to 0.5 mgd from the well. At this rate of pumping the water levels in the various wells recovered a few feet and stabilized.

Following compilation of data from the operation of the permanent works described above, Camp, Dresser & McKee stated:

"As a result of our studies of the pumping and water level data at this source, we estimate the safe continuous yield of this supply during periods of severe drought at about 0.5 mgd, or 350 gpm. This means that over extended periods of time during dry seasons the well should not be operated more than one-half the time. It may, however, be operated for several days at the full pumping capacity when needed to meet exceptionally heavy demands."

Taylor Farm Supply

The estimated combined safe yield of the Jordan Avenue and Williams Farm supplies is 2.0 mgd, which in 1955 was considered adequate to serve the needs of the District, including the Naval Air Station. During that year, however, the District was approached by the Federal Government relative to a new U. S. Air Force installation to be located in Topsham. It was estimated that this installation would use 0.3 to 0.5 mgd. It was considered essential by the District that an additional supply be developed if the Air Force installation was to be served. Accordingly, Camp, Dresser & McKee was authorized to make an engineering investigation and report on developing a ground-water supply at the Taylor Farm site. The location of the Taylor Farm site is shown on Fig. 3.

The R. E. Chapman Company had previously done considerable exploratory work at the Taylor Farm site in 1954, installing seventeen 2″ test wells along the southwest bank of the Androscoggin River (See Fig. 4 for location). Several of the 2-in test wells were jetted to depths of more than 100 ft, at which depths sand and gravel deposits were encountered which yielded water freely when pumped.

The R. E. Chapman Company had also conducted a continuous pumping test on a temporary 8-in test well at the Taylor Farm site. The 8-in test well was 120 feet deep and contained 20 ft of well screen. The test well was pumped at the rate of 725 gpm for a period of 14 days ending on January 25, 1955. The water level in the various observation wells stabilized after about 9 days of pumping and remained relatively level during the remainder of the test, being influenced only by the rise and fall of the Androscoggin River. During the test, the water level in an observation well 3 ft distant from the 8-in test well drew down a total of approximately 18.5 ft. After completion of pumping, com-

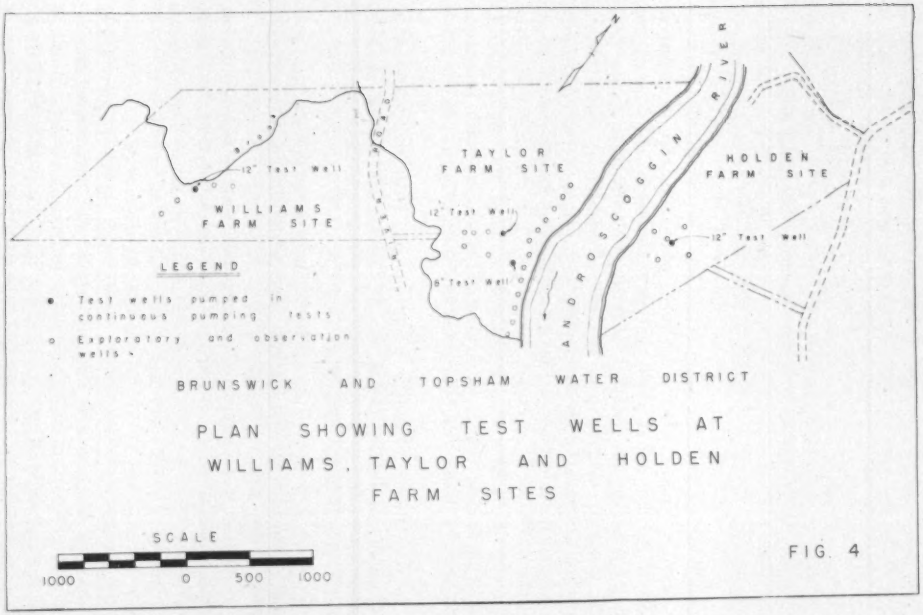

N
ANDROSCOGGIN RIVER
TAYLOR FARM SITE
HOLDEN FARM SITE
WILLIAMS FARM SITE
12" Test Well
12" Test Well
12" Test Well
8" Test Well
LEGEND
Test wells pumped in continuous pumping tests
Exploratory and observation wells
BRUNSWICK AND TOPSHAM WATER DISTRICT
PLAN SHOWING TEST WELLS AT WILLIAMS, TAYLOR AND HOLDEN FARM SITES
SCALE
1000 0 500 1000

FIG. 4

plete recovery of the 8-in well and the observation wells took place within 24 hours.

The Consulting Engineers reviewed this data. Analyses of water samples taken from the 2″ exploratory wells and from the 8-in test well during the 14-day pumping test indicated water of good quality for drinking and other domestic uses. Likewise, analyses of water samples taken during the long-term pumping tests on the Durham Road and Williams Farm supplies previously described also indicated water of good quality. However, the engineers were not satisfied. They agreed that the 14-day test at the Taylor Farm site indicated that a supply of 2 mgd could be developed at this site, but felt that a much longer test would be required to obtain reliable data on water quality. Their reasons were contained in a letter to the District dated May 26, 1955, quoted in part below:

> "The previous pumping test on the 8-in well, while adequate from the standpoint of determining the available yield of the area, was not extensive enough to give reliable information on the probable quality of the water which could be pumped from this source over a long period of time. There will be a substantial amount of infiltration from the Androscoggin River into the proposed well. The water in the Androscoggin River at times contains a high concentration of sulfite liquor. These wastes, to a large extent, remain in solution, and may under certain circumstances infiltrate into ground-water supply, causing taste and odor problems. In order to reliably determine whether there will be a problem from this source at the proposed Taylor Farm wells, a pumping test at the full proposed 2 mgd design rate should be conducted for a period of at least 2 months.
>
> "Previous test-well operations on the Taylor Farm in the general area of the existing test wells have revealed the presence of high concentrations of iron in the ground water. This iron has generally been found at much shallower depths than the present test wells, and it may well be that this source of iron will be effectively sealed off by a barrier of clay which is generally prevalent in this area at a somewhat lower depth. However, only extended pumping will give a reliable indication as to whether iron is likely to be a problem at this supply."

Accordingly, arrangements were made for a pumping test of about 3 months' duration at the Taylor Farm site. It was decided to install a second test well of 12-in size for pumping purposes. In order to find the best location for the 12-in test well,

five additional 2-in test wells were installed running inland from the river bank. It was found that water from those wells farthest inland had a high iron content, in excess of 1 ppm. It was therefore decided to install the 12-in test well at the location of the test well nearest the river, in order to remain as far away as practicable from the iron-bearing area. The 8-in test well was located about 170 feet from the river and the 12-in well about 300 ft from the river as shown on Fig. 4.

A temporary electric power line was brought in from River Road to the sites of the 8-in and 12-in test wells. Both wells were equipped with electric motor-driven turbine pumps, the motors and electrical control equipment being housed in temporary wooden buildings. Temporary discharge lines were installed from each well to the river. Pumping of both wells was started on August 20, 1955, and carried on continuously until November 14, 1955. The 8-in well was pumped uniformly at 675 gpm during the entire test, and the 12-in well at 390 gpm during the entire test.

The water level in the various wells stabilized in a period of about 2 weeks and remained more or less constant during the remainder of the test, except for fluctuations caused by changes in the level of the Androscoggin River. The maximum drawdown in an observation well 3 ft distant from the 8-in test well was 19 ft. The maximum drawdown in an observation well about 3 ft from the 12-in well was 33 ft. After pumping ended, all of the wells recovered to the original static level in less than 24 hours.

A total of seventeen water analyses was made on each well during the continuous pumping test. The iron and manganese content in the water from each well increased materially over that of the water initially pumped, but these constituents did not tend to increase during the last six weeks' pumping and remained within the allowable limit of 0.3 ppm (parts per million) for combined iron and manganese established by the U. S. Public Health Service Standards for Drinking Water. The color of the water from the 8-in test well increased from 0 to 10 ppm after one month's pumping, but showed no increasing trend during the remainder of the pumping test. The USPHS allowable limit for color is 20 ppm. The water ranged from 60 to 70 ppm in hardness, which is more than twice the hardness of the Jordan Avenue

supply, but nevertheless is not considered excessive. The analyses also showed that the water would not be very corrosive to metallic pipes.

In order to show that water was infiltrating from the direction of the Androscoggin River to the wells being pumped, a concentrated solution of common salt (sodium chloride) was added to an observation well located between the river and the 8-in test well being pumped. The observation well was located about 70 ft from the river and the 8-in test well about 170 ft from the river, the distance between the two wells, therefore, being about 100 ft. After adding the salt solution to the observation well, hourly samples were taken from the 8-in pumped well and analyzed for chloride content. The increase in chlorides was quite pronounced, the maximum occurring about 16 hours after the salt solution was placed in the observation well.

The taste and odor of the water as discharged from the 8-in test well was disagreeable during most of the pumping test. Tests were therefore conducted in the field at the well site to determine what treatment would be required to improve the taste of the water to acceptable standards. Four principal types of treatment were investigated: (1) aeration, (2) chlorination, (3) chloramine treatment, and (4) chlorine dioxide. It was found that chlorination alone with a contact period of about 30 minutes would produce satisfactory results.

Various alternate plans were studied by the consultants for construction of the permanent works at the Taylor Farm site, one of the big problems being the fact that flood stages have been as high as 14 ft above the ground surface. The recommended plan which was completed in July, 1958, is shown on Fig. 5.

The completed works include two wells, each of which is equipped with a submersible-type turbine pump. The new gravel-packed well at the 8-in test well location is 116 feet deep and contains 20 feet of 18-in Everdur screen. The pump capacity is 1000 gpm. The well at the 12-in test well site is a "developed" well 160 ft deep and contains 25 ft of 12-in Everdur screen. The pump capacity is 440 gpm. An underground concrete vault with watertight cover, containing discharge piping and check valve, is constructed over each well. Neither well is vented, and each well

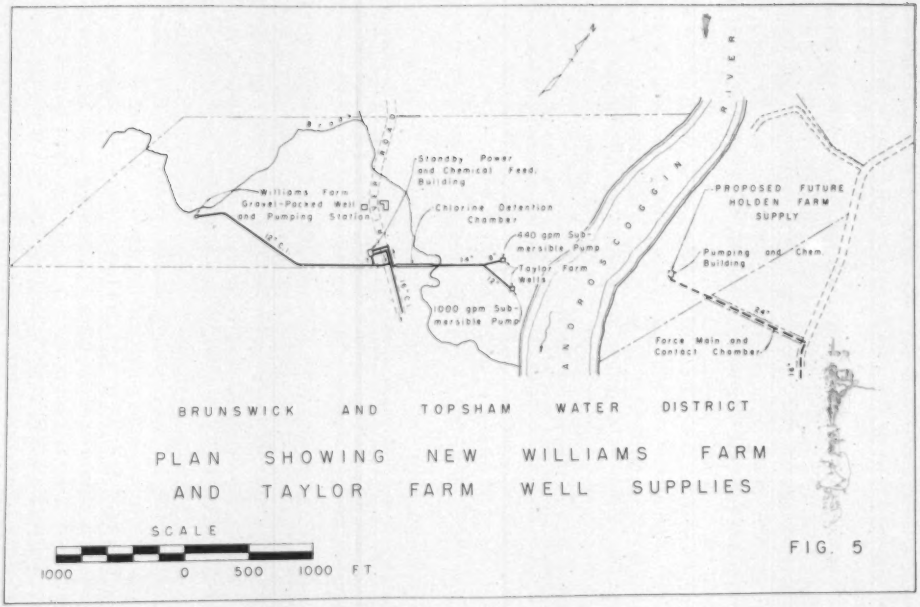

Standby Power and Chemical Feed Building
Williams Farm Gravel-Packed Well and Pumping Station
Chlorine Detention Chamber
640 gpm Submersible Pump
Taylor Farm Wells
1000 gpm Submersible Pump
ANDROSCOGGIN RIVER
PROPOSED FUTURE HOLDEN FARM SUPPLY
Pumping and Chem. Building
Force Main and Contact Chamber
BRUNSWICK AND TOPSHAM WATER DISTRICT
PLAN SHOWING NEW WILLIAMS FARM AND TAYLOR FARM WELL SUPPLIES
SCALE
1000 0 500 1000 FT.

FIG. 5

is watertight against the entrance of surface water should the vault become flooded.

Water from the two wells discharges into a chlorine contact chamber consisting of 320 ft of 54-in prestressed concrete cylinder pipe. After passing through the chlorine contact chamber, the water is conducted to the water hammer control system located in the basement of the River Road Building. This system consists of an air tank, surge tank, air compressors, and the necessary surge tank water level controls. Under normal conditions all water from the Taylor Farm wells and the Williams Farm supply passes through the surge tank. Due to the remoteness of these supplies and the characteristics of the system, stopping and starting surges would be extremely high if no provisions were made to reduce them. Tests conducted at the time the station was put into service showed the effectiveness of the system installed. When pumping all wells at a total rate of 2,100 gpm the running pressure was 88 psi. With a simultaneous shutdown of all three pumps, the pressure dropped to a low of 48 psi, and the returning surge did not exceed 92 psi.

In addition to the water hammer control system discussed above, the River Road Building (labelled Taylor Farm Stand-by Power and Chemical Feed Building on Fig. 5) contains a diesel-generator set to provide power for operating either of the Taylor Farm pumps and for station service during times of power outages, chlorination equipment, fluoridation equipment, and the electrical and metering equipment for the Taylor Farm supply.

Standby power was installed for this supply because the normal auxiliary source of water, the Jordan Avenue Station, is not capable of operation during major floods. Since it is entirely possible to have a power outage at the Taylor Farm site at the same time the Jordan Avenue Station is kept out of service by high water, a stand-by power unit at the River Road Building was deemed necessary to insure an adequate supply of water to the District at all times.

The chlorinator is paced by a Dall Flow Tube so the rate of chlorine dosage remains the same for all flows. The chlorine solution is pumped to an injection point located on the pipeline just upstream of the chlorine detention chamber. The chlorine dosage required to produce a palatable water is normally about 2.3 ppm.

This dosage results in a chlorine residual of about 0.50 ppm at the River Road Building, which is further reduced to less than 0.2 ppm before the water enters the distribution system.

Provisions have been made in the layout of the building for additional, or different, treatment of the water if in the future the chlorination treatment now being used proves ineffective against the taste and odor problem. Space has been provided for ammonia or chlorine dioxide feeding. Station piping is arranged such that it may be easily remodeled if the construction of a filter plant is ever required.

Holden Farm Supply

During the period of negotiations between the U. S. Government and the District relative to construction of the Taylor Farm supply described above, the District acquired the Holden Farm property directly across the Androscoggin River from the Taylor Farm site (See Fig. 3 for location). Previous exploratory work on the Holden Farm property by the R. E. Chapman Company for the Government had revealed the possibility of developing an excellent ground-water supply at this location.

The consultants were authorized to make an investigation and report on this site for the District. Accordingly, a long-term continuous pumping test similar to those described previously at the other sites was conducted at the Holden Farm location in the summer and early fall of 1956.

The test well pumped was a 12″ well, 138 feet deep, containing 25 feet of well screen. The location of the 12″ well and six observation wells on the Holden Farm site is shown on Fig. 4. The well was pumped continuously at the rate of 900 gpm from August 1, 1956, to October 1, 1956. The maximum drawdown in the 12-in well was 6.4 feet, equivalent to 140 gpm per foot of drawdown. Within a period of three hours after starting pumping, the water level in the 12-in well and the observation wells reached a stabilized condition, and thereafter the water level in the wells rose and fell strictly in accordance with variations in the level of the Androscoggin River. The water level in an observation well across the river at the Taylor Farm site was lowered a maximum of 1.6 feet during the test, indicating that both the Taylor Farm and Holden Farm wells are located in the

same aquifer. At the conclusion of the pumping test the water level in all of the wells recovered completely in a period of about 2 hours.

A total of 9 water samples were collected during the pumping test and analyzed by the Maine Department of Health and Welfare. These analyses revealed that the water was similar in quality to that obtained from the Taylor Farm Supply, except that the combined iron and manganese content was less than 0.1 ppm. The water contained a noticeable taste and odor, similar to that of the Taylor Farm well water, but experience at the Taylor Farm supply shows that the taste and odor can be successfully removed by chlorination.

The Consultants stated the following in their report on the Holden Farm supply:

> "The pumping test described above revealed that the subsurface formation or aquifer being pumped is extremely permeable, and can be developed for exceptionally large quantities of ground water. In our opinion, this source could be developed for a supply of several million gallons per day, a supply greater than the foreseeable demands of the District. The watershed feeding this source is in effect the entire Androscoggin River, and the yield of the supply is limited principally by the number and size of permanent wells installed. From a practical viewpoint the capacity of individual wells to be installed at this site should be limited to about 2 mgd each.
>
> "Although this source could be developed for several million gallons per day of ground water, it should be kept in mind that with such development the quality of the water might suffer seriously."

The Consultants went on to state that with heavy use of the Holden Farm supply, chemical treatment of the water for taste and odor removal might prove to be insufficient, and made preliminary designs for possible future activated carbon pressure filters.

The project as recommended by the consultants is shown on Fig. 5. The proposed pumping station would contain a 1400 gpm deep well turbine pump driven by a 100 HP electric motor. The operating floor of the pumping station would be 18″ above the maximum flood level of 1936, or about 8½ feet above the existing ground. The building would contain chemical feeding facil-

ities similar to those provided at the Taylor Farm supply described above, electrical control equipment, and space for the possible future installation of a standby diesel power unit. Water would be pumped from the well through a 24-in force main to Lisbon Falls Road, which would provide a chlorine contact period of about 21 minutes at the 1400 gpm pumping rate, and thence through a new 16-inch main to the distribution system.

The proposed Holden Farm supply works described above were considered to be an alternate to development of the Taylor Farm supply. Negotiations with the Government led to the decision to construct the Taylor Farm supply, as has been described hereinbefore. The Holden Farm supply, therefore, will probably not be developed within the foreseeable future, but provides a valuable potential source of supply to the District.

Cost of Construction

The total cost of the improvements to the Jordan Avenue supply, the new Williams Farm supply, and the new Taylor Farm supply was $548,064.07, as shown in Table 1.

The type of contract which was negotiated with the Federal Government will be of interest to many of you here. In the case of the Jordan Avenue and Williams Farm projects, which were necessitated primarily by expansion of the Naval Base, the District felt that because of the uncertain duration of Naval Base activities and the attendant unnatural population increase, it would be dangerous for the District to invest the entire amount of capital needed to develop these supplies. It was felt that the increased financial load from a large bond issue would result subsequently in a rate increase which would be unfair to the old customers of the District. Therefore, we sought, and were successful in obtaining, a contract in which the Government advanced, under a "connection-charge" type of contract, approximately ¾ of the total cost of construction of these supply facilities. This advance will be refunded to the Government at the rate of 27½ per cent of each quarterly bill until the total advance is liquidated.

In the case of the Taylor Farm project, which was necessitated primarily because of the Air Force installation in Topsham, the same procedure was followed.

The amounts advanced by the Government for each of these three projects are shown in Table 1. It will be noted that of the total cost of $548,064.07, the Government advanced $401,300.00.

TABLE 1.—Basis of Government-District Cost Participation

Name of supply	Total cost	Government share	Water-District share
Jordan Ave. Well Field No. 2 and Pumping Station	$141,299.29	$ 96,500.00	$ 44,799.29
Williams Farm Supply	179,675.27	142,800.00	36,875.27
Taylor Farm Supply	227,089.51	162,000.00	65,089.51
Totals	$548,064.07	$401,300.00	$146,764.07

The estimated cost of development of the proposed future Holden Farm supply, including an allowance of 20% for contingencies and engineering, is $246,000, which includes 1300 feet of 24-in and 9700 feet of 16-in cast iron force main.

Summary

1. The rapid expansion of U. S. Government military facilities at Brunswick and Topsham resulted in a major increase in population and water consumption and required prompt construction of new water supply facilities by the Water District.

2. Renovations and improvements to the existing well field at Jordan Avenue increased the dependable yield from 1.0 to 1.5 mgd, and showed clearly that keeping wells and suction system in clean condition pays dividends in increased pumping capacity, decreased pumping costs, and improved water quality.

3. The value of well conceived, carefully conducted, long-term pumping tests, when conditions call for such tests, has been well demonstrated during the District's water supply expansion program. Such tests enabled the District to "weed out" a poor producer (Durham Road Well) from the initial program and to obtain reliable information on yield and water quality of the two

new supplies developed (Williams Farm and Taylor Farm supplies.)

4. Long-term pumping tests at the Taylor Farm and Holden Farm sites on opposite sides of the Androscoggin River confirmed that infiltration of river water containing dissolved organic matter would cause taste and odor problems. Field experiments during the pumping tests and subsequent operating experience at the completed Taylor Farm works revealed that the taste and odor problem may be controlled by chlorination.

5. The District has increased the estimated safe yield of its ground-water supplies from 1 mgd to 4 mgd, and has a potential of several mgd additional at the Holden Farm site for possible future development. The total cost of the improvements to the Jordan Avenue supply, the new Williams Farm supply and the new Taylor Farm supply was $548,164. The U. S. Government contributed $401,300 of this cost, which will be reimbursed to the Government by the District at the rate of 27½% of the quarterly bills until liquidated.

Acknowledgments

Metcalf & Eddy were consultants on the improvements to the existing Jordan Avenue supply. The new Jordan Avenue 2½" wells were installed by the D. L. Maher Company of Woburn, Massachusetts. Paul McLellan Co. of Portland, Maine, was general contractor on the pumping station improvements.

Camp, Dresser & McKee were consultants on the new ground-water supplies at the Williams Farm and Taylor Farm sites and on the proposed future Holden Farm supply. R. E. Chapman Company of Oakdale, Massachusetts, installed all of the wells in connection with these projects. Sheldon S. Grant of Raymond, Maine, was general contractor on the Williams Farm pumping station and force main. Samuel Aceto & Co. of Portland was general contractor on the Taylor Farm project for the building work and outside piping. Sanders Construction Company of Portland was general contractor on the Taylor Farm project for equipment and electrical work.

BURLINGTON WATER QUALITY AND TREATMENT-PLANT OPERATION

BY EDWARD L. TRACY*

[*Read September 23, 1959.*]

This study of Burlington water quality and treatment-plant operation started with a question from the Burlington Water Superintendent. In January, 1958, Mr. Francis O'Brien came into the Bureau of Environmental Sanitation, State Department of Health, and asked if it was necessary to use alum in the sedimentation and filtration process. Coming to me unexpectedly, this was like a question that a student asks in a classroom that causes the teacher to say, "That is an excellent question. We will discuss it tomorrow when the class meets again." Like the teacher, I said, "That is a good question," and then began stalling for time to figure out an answer.

The Burlington Water Department had installed a new 24" intake pipe from Lake Champlain in 1957 and had been taking water through it since July 14, 1957. The intake is 6,600 feet out from shore in water 60 feet deep. This water-supply improvement, together with a pumping-plant addition, had been designed by Whitman and Howard, Consulting Engineers. The intake location had been selected after studies and sample analyses in 1954.

Mr. O'Brien observed that from July, 1957, to January, 1958, the water coming into the plant had been very clear, with turbidity of only 5 to 15 parts per million. It had also been observed during this time that there had been less dirt retained in the sedimentation basin and on the filters, and that he believed a considerable portion of the floc was being carried onto the filters. The Burlington filter plant had been in operation since 1908 with alum used as a coagulant. During that fifty-year period the water had been coming through an intake nearly twice as far out from the water plant but in 26 feet of water.

The first step taken by the State Department of Health, to

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help me arrive at an answer to the question about the use of alum, was a series of detailed analyses of raw water and coagulation tests. The average alum dosage for many years had been 1 grain per gallon. In our laboratory to 200 milliliter samples we added alum dosages of ½ grain, ¾, 1, 2, 3 and 4 grains per gallon and stirred them mechanically. The ½ grain dosage produced no floc in the Lake Champlain water from the new intake. The ¾ grain dosage produced a very fine floc that would not settle. The 1 grain dosage produced a fine floc that settled slowly. The 2, 3 and 4 grain dosages produced the same results as 1 grain. None of the floc formations were considered satisfactory. The addition of a very small amount of lime resulted in a large-particle, settleable floc. We concluded that the incoming raw water was too clean and clear to form a floc without a coagulant aid.

The analysis of raw water on January 30, 1958 showed color to be 10 ppm. Turbidity was 5 ppm; pH was 7.5; albuminoid nitrogen was .084 ppm; ammonia was .030 ppm; nitrites were .00 and nitrates were .2 ppm. Hardness was 59 ppm. This obviously was a water that required only chlorination to render it potable. For years the chlorine dosage at the filter plant had been such as to produce a 0.15 ppm chlorine residual in the finished water. Mr. O'Brien increased the chlorine dosage to 1.1 ppm going into the sedimentation basin, resulting in a chlorine residual of 0.5 ppm on the filters. Post-chlorination was set at 0.2 ppm giving a chlorine residual of 0.35 ppm leaving the plant. We agreed that the use of alum could be discontinued until water turbidity, color and organic content increased enough to indicate a need for it. At that time it was expected that the need would come with the Spring runoff of tributary rivers. It was proposed that a schedule of treatment plant operation could be based on the expected variations.

The January observations were reported to Paul Howard of Whitman and Howard so that he would be aware of the treatment plant operation changes that were being approved by the State Department of Health. Mr. Howard replied that although he had expected better quality water through the new intake, he had not anticipated enough improvement to discontinue the use of alum. He expected that Spring runoff of the Winooski

River would require conventional coagulation and filtration and that increased chlorine dosages might cause customer complaints.

Additional complete analyses were made February 7, April 15, June 3, July 15, August 6, September 29, November 25, 1958 and February 27, 1959. None of these analyses showed need for coagulation. The Spring runoff of tributary rivers did not materially change the characteristics of the raw water as had been expected and the organic content did not increase during the summer when the temperature went up to 68 degrees. The water treatment plant had operated during the period since January, 1958 without alum or any coagulant. The higher chlorine dosage was continued and the higher chlorine residual spread to the extremes of the distribution system. We must admit that we were a little surprised that this occured without any objectionable chlorine or chlorine combination taste being detected or reported. Reports on laboratory findings had been sent to Francis O'Brien after each of the sampling dates given above, with continued approval for operation without alum.

The average of all these analyses showed that turbidity had not averaged over 10 ppm; color averaged 10 ppm; pH was 7.3; hardness was 63 ppm; chlorides were 4.8 ppm; albuminoid nitrogen was .057 ppm; ammonia was .033 ppm; nitrites were .00; nitrates were .19 ppm; iron was .03 ppm; calcium was 39 ppm; magnesium was 24 ppm; sodium was 2.3 ppm; total alkalinity was 46.3 ppm and total solids were 115.7 ppm.

In a summary report from the State Department of Health to Mr. Francis O'Brien in April, 1959, the conclusions stated were:

> "The State Department of Health believes the Lake Champlain water processed in the Burlington treatment plant and delivered through the water distribution system has been adequately treated and the quality of water throughout the city has been excellent. The State Department of Health has not found any necessity, from January, 1958 to April, 1959, for the use of a coagulant, such as alum which was used for many years prior to January, 1958. We believe that the water taken in through the new 1957 intake has been superior in quality to the average water from the old intake."

The State Department of Health thanked Mr. Francis O'Brien and members of the Burlington Water Department staff

for their cooperation during the period of the water study. In addition the State Department of Health commended Mr. O'Brien and the personnel under his supervision for the outstandingly excellent record of water quality and of water plant operation.

The quality of the Burlington water had not always been as good as now and the relationship between the Water Department and the State Department of Health had not always been on such a friendly cooperative basis, particularly before the construction of the filter plant in 1908. In October 1906 the old State Board of Health met with a representative group of Burlington business men and strongly recommended, as had previously been done, that a sand filter be constructed. A month later, when the building of a filter plant had been voted, a WARNING NOTICE was published in the daily paper, the Free Press, as follows: "Because water now furnished by the City of Burlington is so contaminated, unwholesome and impure that the use thereof endangers the health of persons drinking it, all persons are hereby warned not to use it without first boiling or otherwise purifying it, until the danger is removed by the new filtration plant. Per order of the State Board of Health." This, of course, was before the beginning of chlorination.

The City of Burlington was incorporated by the State Legislature in November, 1864, and this was ratified by town meeting in January, 1865. On October 1, 1866, the city bought the Aqueduct Company. This water system consisted of a 75,000 gallon underground reservoir fed by springs and said to be "similar to Newton and Brookline." The distribution system had 200 services and it is interesting to note that a paper read by Frank H. Crandall to the New England Water Works Association, at the Convention of September, 1895, held in Burlington, stated that water meters were in use "before the water works were purchased by the city."

To augment this inadequate system, the city decided to construct the first Lake Champlain water intake in 1867, after considerable debate on the relative merits of river water and lake water. The intake extended only 50 feet into the lake. The first open reservoir was built at the present location near the University of Vermont at nearly the highest elevation in the city.

The 2,236,000-gallon reservoir at that time cost only $9,000. Near the intake at the lake shore a Worthington pump with a capacity of 750,000 gallons per day was installed as well as a new main pipe from the pumping station to the reservoir. The first water was pumped Christmas Day, 1867. This was the beginning of the present water system.

Water pollution and typhoid fever were the principal troubles in the following years. The main sewer outlet was less than one-half mile from the water intake. Health officials complained about the hazard and in 1874 Dr. Grinnell, the health officer, recommended extending the intake pipe out into the lake beyond the breakwater. It wasn't until nineteen years later that the intake was moved out away from the shore. Meanwhile in 1888 a 4,000,000 gallon reservoir was built beside the first one and the old one was remodeled and enlarged. At that time a 4,000,000 gallon reservoir, with a concrete bottom and slopes of brick laid in cement with the upper one third of the slope paved with granite blocks laid in gravel, was built for only $23,000. These reservoirs, in excellent condition, are in use now, 71 years later.

In 1894 the intake was extended into the lake 11,300 feet with 24" cast iron pipe. A high service system with a 163,000 gallon tank was added to serve buildings at elevations too high for the reservoirs. The new intake, northwest of the pumping station, was located at Appletree Reef in Lake Champlain at a depth of 26 feet. This intake pipe was installed by J. G. Falion of Evanston, Illinois, for $47,900. The feature was a "Falion ball joint" every 75 feet to provide flexibility. The 75 foot sections were of conventional cast iron pipe with leaded joints. The intake screen was turned up so that its highest point was 14 feet below normal low water. It was relatively shallow water that flowed in through this intake.

Water was pumped from the reservoirs to the high service system by means of a water motor designed and built by W. H. Lang of Burlington. For each 20 gallons passing through the water motor, either flowing to or from the reservoir, 1 gallon of water was forced into the high service system. An overflow pipe from the high service tank back to the reservoirs, returned any surplus water pumped into the high service system. On this high service system a meter was required on every service to a

building having a sewer connection. For the rest of the city distribution system, a meter was required on all services larger than ½ inch.

By 1895 the water consumption taxed the capacity of the original 750,000 gallon pump and a second similar pump was installed. By 1900, a population of 18,640 was being supplied with an average of 1,057,000 gallons of water per day. This was a per capita rate of 54 gallons.

All published reports appear to indicate that there was a decrease in typhoid fever incidence after the intake was extended in 1894. At the September, 1895 meeting of the New England Water Works Association in Burlington there was a lengthy discussion, following Mr. Crandall's paper, of the extension of water intakes to safer locations. Mr. Hazen compared the Burlington water intake to several systems on the Great Lakes, particularly Chicago. He stated that water intakes should be "far enough from sewer outlets so that sewage will have no great effect on the health of the city." He went into great detail to evaluate the deaths from typhoid fever to justify the extension of water intakes. This was evidently an early attempt to express public health in terms of economics.

The reduction in typhoid fever in Burlington could not have lasted very long as reports indicate that in 1904 the morbidity and mortality rate was again rising. In 1905 the Mayor recommended a filtration plant and in 1906 the State Board of Health forced action on the problem as was described earlier in this paper. In 1907 construction started and in April, 1908 the new rapid sand filter went into operation. At that time the water system was serving 20,700 people with an average of 55 gallons per capita per day. The use of hypochlorite of lime is believed to have been started about 1914. In November, 1920 the intake pipe became nearly clogged and an opening was cut in the 24″ pipe about ¾ mile out from shore. This trouble was not corrected until 1938 when Superintendent Joseph Moore had the pipe cleaned and repaired and water again flowed from the intake 11,300 feet from the water plant. Gas chlorination had been started in 1920. At that time the four filter units were each handling from 600,000 to 750,000 gallons per day; a total of 2,400,000 to 3,000,000 gallons. During the summer the filter

capacity was considerably overtaxed. In 1938 a diesel motor for pumping was installed for operating cost reduction. In 1948 the filter capacity doubled by the addition of four more units. The coagulation basin was doubled and equipped with mechanical mixers. A curtain wall to provide a three-foot insulating air space was built completely surrounding the filters and the coagulation basin.

In 1954 Whitman and Howard began their studies to enlarge the intake capacity. For several summers prior to that year it had been necessary during peak demand to use auxiliary pumps, either fire trucks or Civilian Defense pumpers, to get additional water into the plant because the old 24 inch intake couldn't supply enough water These auxiliary pumps had to take water of inferior quality adjacent to the water plant dock. Although the filtration and chlorination plant still produced a good finished water it was obvious that the makeshift arrangement should not continue. The maximum daily consumption reached 5,070,000 gallons on July 18, 1953 and 5,280,000 gallons on August 10, 1955. Whitman and Howard had found that the maximum delivery of the intake pipe was about 4,000,000 gallons per day. The proper maximum capacity of the filter plant was considered to be 5,000,000 gallons per day. The average daily consumption of water in 1954-55 was 2,960,000 gallons per day.

Four plans for increasing intake capacity were considered and the final recommendation in August, 1955 was to install 6,600 feet of new suction main with the intake located 1,500 feet south of Rock Point in 60 feet of water. Also to cut off the old 24 inch pipe line and relocate the intake at a point opposite the new pipe intake. This was between the new intake and Rock Point. In addition a new suction pumping station on the lake side of the existing pumping station was proposed to provide a total pumping capacity of 10,000,000 gallons per day. Cost of this plan was estimated at $290,000.

The recommendation was accepted by the Water Commissioners and the installation of the 24 inch mechanical joint cast iron pipe was started in the Summer of 1956. The job was finished in 1957 and operation of the combined new and old intake and pumping station started July 14, 1957. Operation of the filter plant continued the same as in previous years with an

alum dosage of 1 grain per gallon and chlorination to give a residual of at least 0.15 ppm in the water leaving the plant until January, 1958. Then the use of alum was discontinued and the chlorination rate was increased as described in the first part of this paper. The analyses from January, 1958 to February, 1959 showed no great variations in the characteristics of the water. The largest spread was in solids from a minimum of 35.5 ppm to a maximum of 284 ppm. Turbidity and color varied hardly at all which was the greatest surprise of the series of tests. The maximum turbidity during the 15 months period was 15 ppm and the maximum color was 15 ppm. The minimum in each case was 0. The variation in hardness and pH was very slight. Chlorides varied from 2.5 ppm to 6.1 ppm. Albuminoid nitrogen varied from 0.006 ppm to 0.132 ppm with an average of 0.057 ppm. Ammonia varied from 0.000 ppm to 0.114 ppm with an average of 0.033 ppm. Nitrites were always absent and nitrates varied from 0.0 ppm to 0.5 ppm with an average of 0.19 ppm. Sodium varied from 1.9 ppm to 2.5 ppm. Total alkalinity varied from 40.6 ppm to 56 ppm.

Additional samples of raw Lake Champlain water were taken at the plant on September 4, 1959. The physical and chemical analyses showed better quality than at any other time in 1958 or 1959. The detailed analysis is as follows: turbidity, 0; color, 5 ppm; pH, 7.5; hardness, 59 ppm; chlorides, 1.9 ppm; albuminoid nitrogen, 0.078 ppm; ammonia, 0.009 ppm; nitrite nitrogen, 0.00 ppm; nitrate nitrogen, 0.0 ppm; calcium, 34 ppm; magnesium, 25 ppm; sodium, 2.5 ppm; iron, 0.02 ppm; total solids, 67 ppm and total alkalinity, 44 ppm. Two bacteriological samples of raw water were taken on the same date. One had MPN 130 coliform organisms and the other had MPN 16 coliform organisms per 100 milliliters. In August and September, 1958, the coliform organism count had been MPN 2,400 per 100 milliliters.

Treated water, delivered through the Burlington distribution system, has been very good in physical and chemical quality for many years as shown by the following tabulation of analyses for which records were readily available.

BURLINGTON WATER FROM DISTRIBUTION SYSTEM AFTER TREATMENT

	Average 1946-50	1952	1953	6/9/55	4/2/57	11/25/58	3/10/59	4/9/59	Average
Sediment	0	0	0	0	0	1	1	1	0
Turbidity	0	0	0	0	0	0	5 ppm	5 ppm	0
Color	0	0	5	0	5	0	5	5	2
Odor	0	0	1	0	0	0	0	0	0
pH	7.3	7.2	7.0	7.1	7.0	7.2	7.1	7.1	7.1
Hardness—ppm	56	64	62	61	60	72	70	64	64
Chlorides—ppm		2.8	3.5	3.4	4.0	5.2	6.5	6.0	4.5
Albuminoid—ppm		.048	.018	.066	.084	.066	.072	.042	.057
Ammonia—ppm		.000	.000	.000	.000	.000	.018	.018	.005
Nitrite—ppm		.00	.00	.00	.00	.00	.00	.00	.00
Nitrate—ppm		.14	.08	.00	.20	.10	.2	.1	.12
Lead—ppm		.02							
Copper—ppm		0.0							
Iron—ppm		0.0				0.0	0.0	0.0	
CO_2—ppm		7.4							
Total Alkalinity—ppm		29.0						38.0	
Total Solids—ppm								250	
Sulphates—ppm		13.0							
Calcium—ppm						40	40	38	
Magnesium—ppm						32	30	26	
Sodium—ppm						3.1	3.4	2.8	

The bacteriological quality of the finished water in the distribution system has improved since the higher rate of chlorination started.

Year	Number of samples analyzed	Percentage of contamination
1952	572	2. %
1953	398	1.6 %
1954	378	3.1 %
1955	381	2.4 %
1956	292	2.9 %
1957	374	3.5 %
1958	392	0.51%
1959 (Jan. to Aug.)	243	0.08%

These figures are based only on samples analyzed by the State Department of Health and includes mostly high service system samples. Such water has been through the open distribution reservoirs where it is protected by additional chlorination because it is potentially subject to recontamination.

During 1958 and 1959 the water system has been serving a City population of 37,000 plus 3,000 at the University of Vermont and 5,000 in South Burlington outside the city. This is a total of 45,000 people. In addition, Burlington has been taking care of the City of Winooski when its water supply system was deficient in 1957, 1958 and 1959. When the Winooski reservoirs get low, water is pumped through a connecting pipe across the Winooski River bridge. About 20,000,000 gallons of water is pumped in a little over 20 days to bring the reservoirs back up to proper level. During that period of time the Burlington water system is supplying a total population of 53,000 besides filling the Winooski reservoirs.

During the fiscal year 1958-59 the water supplied from the Burlington plant was 1,244,677,000 gallons to the 45,000 population. This is an average of 75.7 gallons per capita per day. The two biggest days were in July and in August, 1959, when 6,423,-000 gallons of water were pumped, filtered, chlorinated and delivered through the distribution system. It is believed this could not have been accomplished if it had been necessary to add a coagulant in the sedimentation basin and operate the filters in

the conventional manner as prior to January, 1958. The peak pumping rate reached on these two days was 7,500,000 gallons.

It is the conclusion of the State Department of Health and the Burlington Water Department that the Lake Champlain water taken into the Burlington water plant since January, 1958 does not require the use of alum. We have been pleased to find, during the twenty-one months of study in 1958 and 1959, that the water from the new intake at the 60 foot depth has been of very uniform excellent quality. We know that the water delivered to the consumers has been of excellent quality and now with a chlorine residual throughout the distribution system we have a greater sense of security with the added public health protection. One of the most pleasant things about the whole operation was the uniform excellent water quality and the absence of taste or odor problems and complaints.

A PILOT PLANT FOR WATER TREATMENT AND ITS OPERATION AT MIDDLETON POND, MASSACHUSETTS

BY WILLIAM B. DAVIS* AND H. J. STEINHURST**

[*Read September 23, 1959.*]

INTRODUCTION

The existence of unpolluted watersheds in Massachusetts has resulted in many municipalities obtaining their water supply directly from open storage in which moderate to heavy vegetative growth manifests itself through water of high organic color. This water, albeit acceptable from a biological standpoint, is unacceptable from an esthetic standpoint because of the high color and recurring taste and odor problems. The coliform MPN of these waters is generally less than 50 per 100 ml so that chlorination will yield a water of drinkable quality but nevertheless subject to consumer complaints because of the poor physical appearance. Even so, in areas where these waters exist, the local taxpayers, hardpressed by other demands, have not seen fit to appropriate the necessary funds to improve the physical appearance of the water after repeated recommendations by the State Health Department.

Water consumption is steadily increasing, and a number of municipalities are already approaching the limit of safe yield of their existing permanent sources. It is evident that consideration must be given to the development of new supplies overlooked in the past because of the extremely high color content.

Clearly then, it is none too early to begin intensive studies for treatment of specific sources which must precede the design of actual plants. Unfortunately, much of our knowledge of water treatment remains empirical in nature. Laboratory studies to determine, for example, the best coagulant and the optimum dos-

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age for the treatment of a specific water give useful indications but are rarely conclusive enough to serve as a basis for design of full scale plants. Experience has shown that, as in many chemical engineering processes, pilot plant studies form an essential link between laboratory experiments and final design.

Both laboratory and pilot plant data are largely lacking, so far as concerns possible treatment schemes for many of our existing and proposed water supplies. Accordingly, early in 1958, the Division of Sanitary Engineering of the Massachusetts Department of Public Health determined to begin laboratory treatment studies. Later, the design and construction of a flexible pilot plant capable of providing complete treatment for a flow of 5 gallons per minute was completed. The plant, comprising a flocculation chamber, sedimentation tank, and rapid sand filter, together with necessary chemical feed equipment and appurtenances would form the basis for determining the information necessary to extend the pilot plant results to prototype design.

With the plan to operate at various surface sources throughout the State, an investigation was initiated at Middleton Pond, a source of water supply for Danvers and Middleton, with the thought that filtration might be unnecessary if adequate coagulation and floc removal could be accomplished. The floc not removed in such a process would have to be redissolved by pH adjustment before discharge to the distribution system.

Development of effective treatment of this type would substantially reduce the capital outlay necessary for full-scale treatment. The reader should recall that we are dealing with waters in which adequate bacterial control can be obtained by chlorination.

The data obtained by model studies are applicable to prototype design provided the model and prototype exhibit complete geometric and dynamic similarity. Since dynamic similarity is obtained only if the force systems in the model and prototype are identical, it must be realized that few, if any, model flows portray perfect dynamic similarity with their prototypes. The result is that incomplete similarity can be used only if the accompanying reservations are realized and model data are used with caution. This does not preclude the use of models in hydraulic and sani-

tary engineering studies. On the contrary, significant information can be gained from their use.

The predominant force acting in water treatment facilities is gravity. Viscosity, pressure, surface tension, and elasticity can be neglected without introducing serious errors, provided model surface roughness is kept to a minimum and the model is sufficiently large to permit two dimensional flow to develop. The main problem encountered in the use of models for water treatment design is that floc formation cannot be scaled down. Therefore, the model flocculation chamber and sand filter must be used without a time or velocity scale. The sedimentation studies are valid provided the model is not so small that short circuiting accompanies the flow.

For the design employed herein, these problems were fully realized, and an attempt was made to design the pilot plant for a range of flows covering those which might be encountered in the field without the inaccuracies inherent in large model to prototype scale factors.

Design and Description of the Model

The pilot plant provides coagulation, sedimentation, and filtration. Rapid mixing is accomplished by a series of 180° bends in the flocculation chamber influent pipe. Although the quality of effluent from the sedimentation tank is the key to the value of the process, it was felt that this effluent should be compared with that attainable by filtration. Such a comparison is essential to evaluation of the treatment proposed.

Versatility of design was an important consideration, since the pilot plant studies were to be extended to other surface sources.

The choice of lucite as the material of construction was made on the basis of providing a smooth surface for which the surface roughness would be minimum. Ease of construction and visual observations were also pertinent considerations in the final choice.

Investigation of the surface supplies which might be studied indicated that the maximum daily demand from any of the sources would not exceed 10 MGD. To avoid errors inherent in excessive scale ratios, it was felt that the model to prototype

scale should not exceed 25. The sedimentation tank was therefore designed on the basis of a scale factor of 25 for a prototype discharge of 10 MGD. This corresponds to a minimum model flow of 2 GPM, with an overflow rate of 155 gal/sq ft/day.

Provisions were made for baffle arrangement with the thought that higher flows could be attained without sacrificing sedimentation tank performance. The resulting decrease in the scale factor would enhance the reliability of the results.

The sedimentation tank is illustrated in Figures 1 and 2.

Several baffle positions were provided to overcome short circuiting which might be encouraged due to a high inlet velocity.

The inlet chamber was designed to discharge through a slot 1 inch high extending the full width of the tank. Small baffles can be placed in the inlet chamber to overcome inlet velocity and regulate the discharge of the influent within the tank.

The initial phase of the study was the development of dispersion curves to serve as a basis for any necessary improvements in the sedimentation tank. It was also decided that the dispersion curves could be compared with settling characteristics to determine the significance of salt run tests as a means of predicting the sedimentation performance.

The flocculation chamber was designed to provide a 20-minute detention period for flows ranging from 5 to 7.5 GPM. Provision of an outlet located 6 inches below the top of the chamber allows a fluctuation of water levels permitting the 20-minute detention period to be maintained as the flow varies between the above mentioned limits. For model flows smaller than 5 GPM, the detention time would, of course, increase but this could not be avoided except by using several flocculation chambers. The cost of providing a number of such models was prohibitive.

The flocculator paddles were designed to be installed longitudinally. Provision of several sprocket sizes on the drive shaft of the flocculator motor provided a means to maintain sufficient turbulence to prevent floc sedimentation. The paddle tip velocity with this arrangement will vary from 15 fpm to 72 fpm. The details of the flocculation chamber are shown in Figure 3.

Sand filter models must also be designed without a time or velocity scale. Two feet of sand depth was provided in accord-

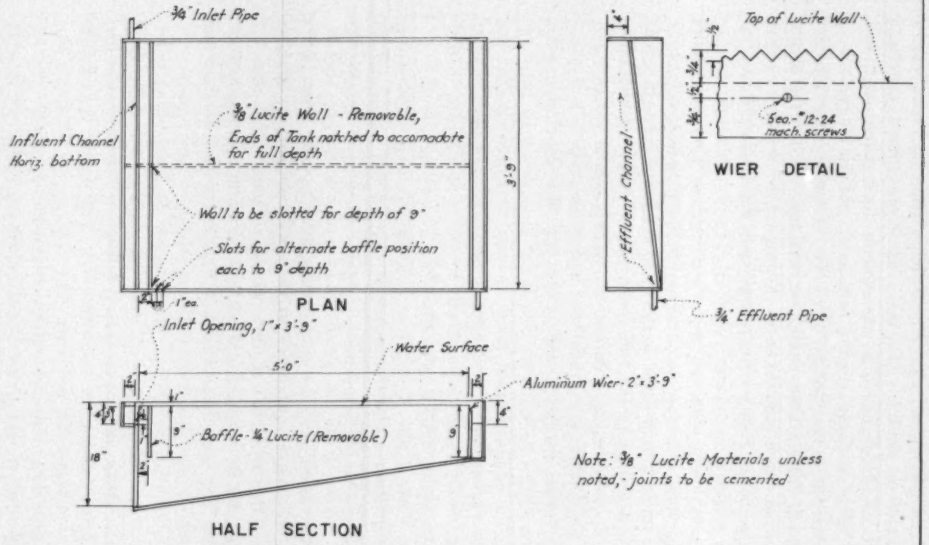


FIG. 1 PILOT PLANT SEDIMENTATION TANK

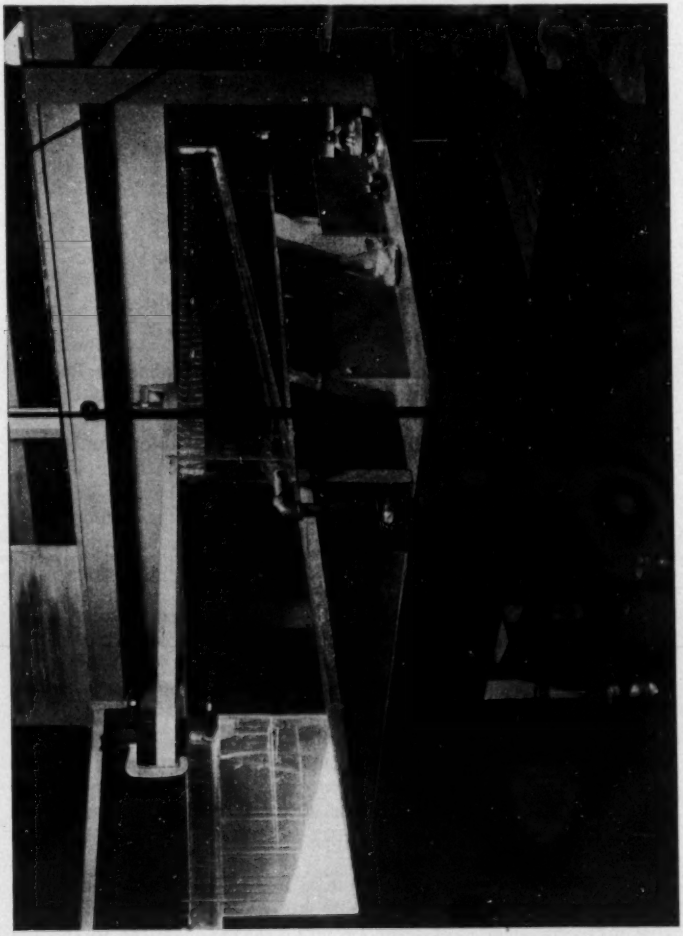


FIG. 2—SEDIMENTATION TANK OPERATION

ance with accepted practice. The lucite plastic casing is two inches in diameter which meant that it could not accept full pilot plant flow. A low lift pump was used to apply a portion of the sedimentation tank effluent to the filter. Sand specifications are as given below:

Hazen's effective size—mm	0.50
Uniformity coefficient	1.40
30% passing size—mm	0.60
50% passing size—mm	0.70
Porosity—percent	42.0
Specific gravity	2.60–2.65
Shape factor	6.5 –7.5

Details of the sand filter are shown in Figure 4.

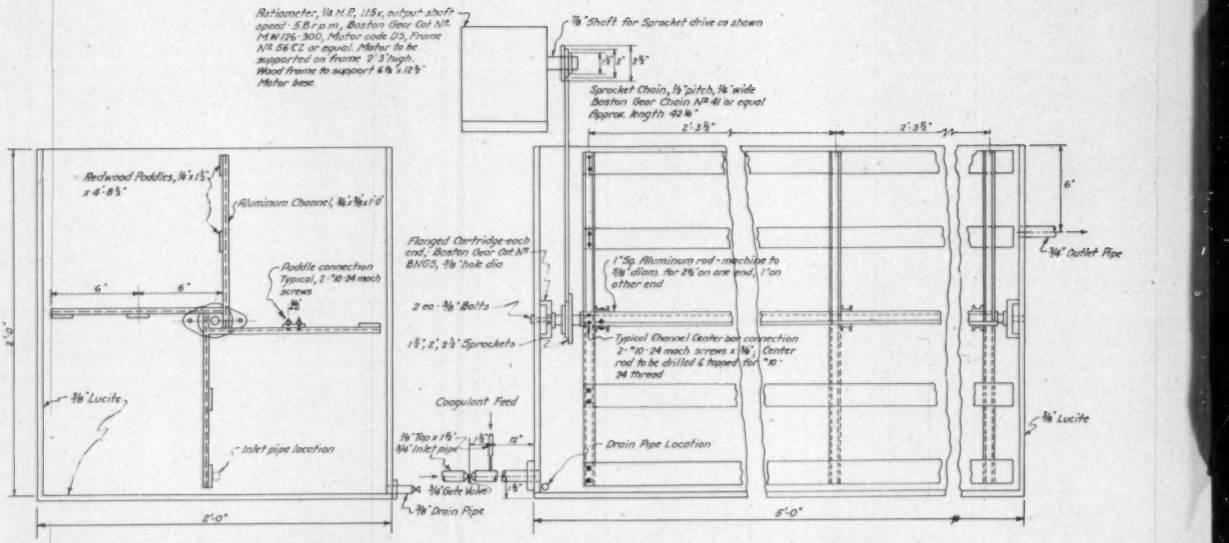
Experimental Procedure

The experiments have been conducted in two phases. The first phase is concerned with the hydraulic performance of the sedimentation tank. The hydraulic performance of a sedimentation basin may be determined by introducing a suitable quantity of tracer material in the influent to the sedimentation basin and measuring the dispersion of the tracer throughout the tank contents. The resulting time-concentration values in the effluent can be plotted to indicate dispersion characteristics of the tank.

In an ideal tank a unit volume entering will leave as the same volume with no dispersion through the tank. The time of passage of this volume is equal to the theoretical displacement time. In an actual tank, however, dispersion of incoming flow will occur due to turbulence, etc. at the inlet.

The time ordinate of the centroid of the resulting dispersion curve has been termed the "flowing-through period". The ratio of this period to the detention time approaches 100% as a maximum and has been termed the "efficiency of displacement". The mode of this curve defines the "most probable detention time". The ratio of the time required for 90% of the tracer to pass through the basin to the time for 10% to pass has been defined as the "dispersion index".

This test is useful but limited in its application to the design of prototype structures from model studies. It is well known that a certain amount of short circuiting occurs in water treat-


Rotameter, ¼ H.P., 115 v, output shaft speed - 5.8 r.p.m., Boston Gear Cat Nr. 141 W/126-300, Motor code D5, Frame Nr. 56 CZ or equal. Motor to be supported on frame 2'-3" high. Wood frame to support 6⅛" x 12⅛" Motor base.
⅞" Shaft for Sprocket drive as shown
Sprocket Chain, ½" pitch, ⅛" wide Boston Gear Chain Nr. 41 or equal Approx. length 42⅛"
2'-3⅛"
2'-3⅛"
6"
⅜" Outlet Pipe
Redwood Paddles, ¾" x 1½" x 4'-8⅝"
Aluminum Channel, ⅛" x 3" x 1'-0"
Paddle connection Typical, 2 - 10-24 mach screws
6"
6"
2'-0"
⅜" Lucite
Inlet pipe location
Flanged Cartridge-each end, Boston Gear Cat Nr. BNGS, ⅜" bolt dia
2 ea - ⅜" Bolts
1⅝", 2", 2⅜" Sprockets
1" Sq. Aluminum rod - machine to ⅝" diam. for 2⅜" on one end, 1" on other end
Typical Channel Center bar connection 2 - #10-24 mach. screws x ⅜", Center rod to be drilled & tapped for #10-24 thread
Coagulant Feed
⅛" Tap x 1⅝" ¼" Inlet pipe
12"
Drain Pipe Location
¾" Gate Valve
⅜" Drain Pipe
2'-0"
5'-0"
⅜" Lucite

END VIEW W/O SPROCKET DRIVE

SIDE ELEVATION

FIG. 3 MODEL FLOCCULATOR

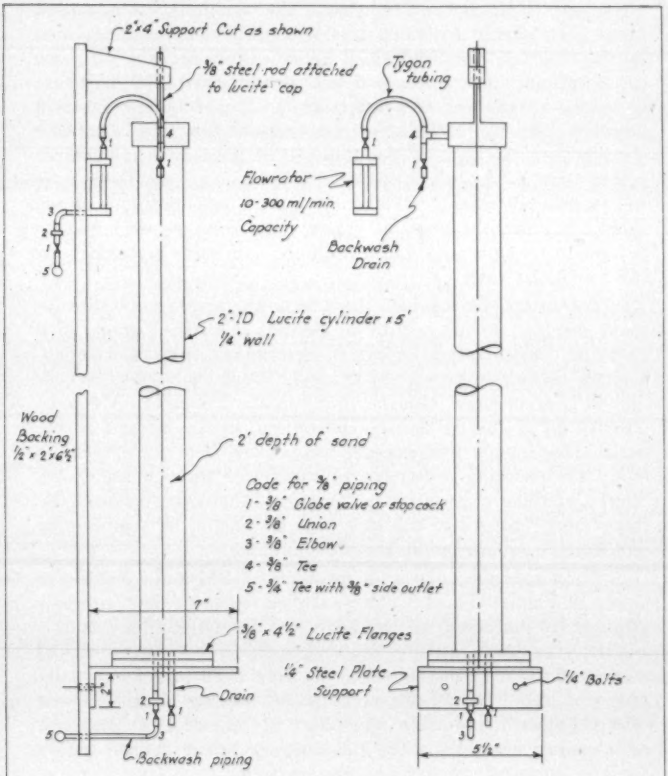


FIG. 4 PILOT PLANT RAPID SAND FILTER

ment units such as the flocculation chamber and sedimentation tank. The degree to which this will occur can be determined by the dispersion test described above. From these results, one can determine the efficiency of inlet design and baffle placement to reduce or minimize this phenomenon. It is to be remembered, however, that the efficiency of the sedimentation tank should be described on the basis of the ability to remove discrete particles rather than by hydraulic indices. In our own case, these tests were used to determine the effect of various baffle positions on the short circuiting phenomenon. From these tests we were able to determine the optimum inlet conditions and baffle placement for this particular tank.

The tracer salt used for these tests was prepared by dissolving 4 grams of sodium nitrite in one liter of distilled water. The resulting concentration of nitrite determined from spectrophotometer measurement was 3400 mg/l. These analyses were conducted in accordance with the 10th edition of Standard Methods. Twenty ml of the tracer were introduced in the influent to the basin. One hundred ml samples were taken from the effluent to study the resulting dispersion of the tracer through the basin. Short circuiting caused by temperature differences between the tracer and contents of the tank was eliminated by maintaining temperature equilibrium of the two solutions.

The dispersion curves for flows of 2 GPM and 3 GPM are shown in Figures 5 and 6. These curves represent the conditions attained by placement of the baffle 18″ from the inlet. At this position the baffle area is approximately three-fourths of the cross-sectional area of the tank. The inlet chamber, as originally proposed, required improvement by introducing a baffle in the inlet chamber to minimize the effect of the velocity head and reduce short circuiting. The dispersion index as defined before for these curves was 5.0 and 5.2 respectively.

Operation at Middleton Pond

An average of about 1.8 MGD is drawn from Middleton Pond to supply Danvers and Middleton.

The yield is augmented by storage in Swan Pond and diversion from Emerson Brook. Existing treatment consists of chlori-

nation, pH adjustment with lime, and fluoridation. The supply is of good bacterial quality but the high color, ranging from about 50 to 100 ppm, is objectionable. Taste and odors from occasional growths of micro-organisms, and fine sediment have also caused complaints. Maximum color usually occurs in August. Temperature in the pond varies between about 34° F. and 75° F.

The pH of the pond is relatively constant over the year at about 6.8, with diurnal variations between pH 6.2 and 6.8. The daily variation appears to be associated with plankton activity.

Since low water temperatures would tend to interfere with sedimentation by reason of increasing water viscosity, it was desirable to include winter operation in the study.

The pilot plant was set up near the Middleton Pond Pumping Station in October, 1958, and operated until early January, 1959.

Raw water was drawn directly from the pond at a depth of about 3 feet. Head conditions and equipment available required that the flow be limited to 2 GPM.

Chemical solutions were fed through small taps in the raw water line ahead of the flocculator. Accurate chemical feed was obtained with a positive displacement pump known as the "Harvard Pump". Rapid mixing was provided by passage through a flow-meter and/or a series of pipe bends.

The setup of the pilot plant is shown in Figures 7, 8 and 9.

Samples for analysis were taken directly from the sedimentation tank for comparison with the filter effluent.

Samples were analyzed for color, pH and for O.T.A. chlorine residual when prechlorination was used. The plant was operated for at least 6 hours under each set of conditions investigated and results were checked by at least 2 additional runs. When color removal was considered satisfactory, more complete analyses were made. Occasional samples of raw and treated water were examined for bacteria and algae.

The flocculator was operated at a peripheral paddle speed of 45 fpm for all tests. The flocculator detention time was 60 minutes.

Detention time in the sedimentation tank was 75 minutes.

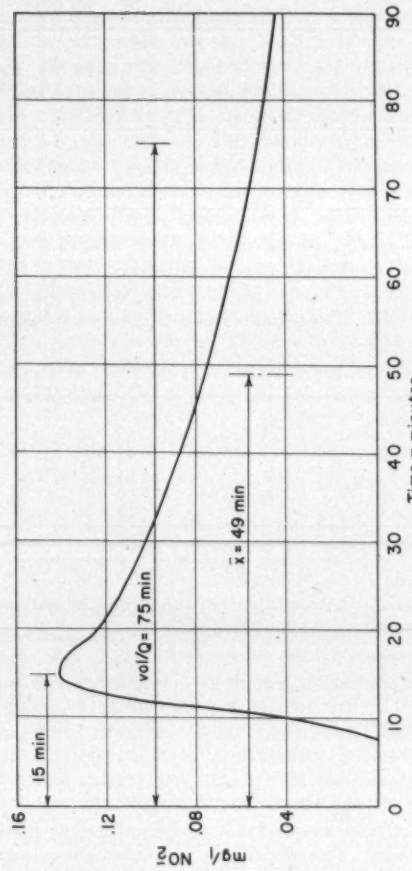


FIG. 5 SEDIMENTATION TANK
DISPERSION CURVE Q=2.0GPM
Baffle Position No. 3

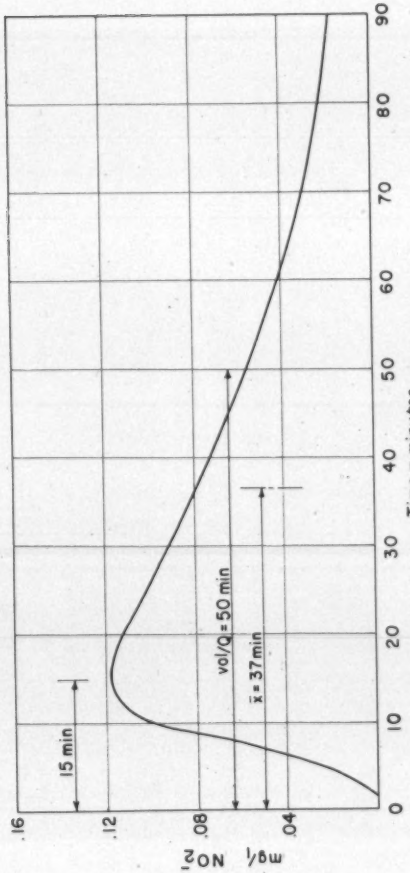


FIG. 6 SEDIMENTATION TANK
DISPERSION CURVE Q = 3.0 GPM
Baffle Position No. 5

FIG. 7.—EQUIPMENT OPERATION

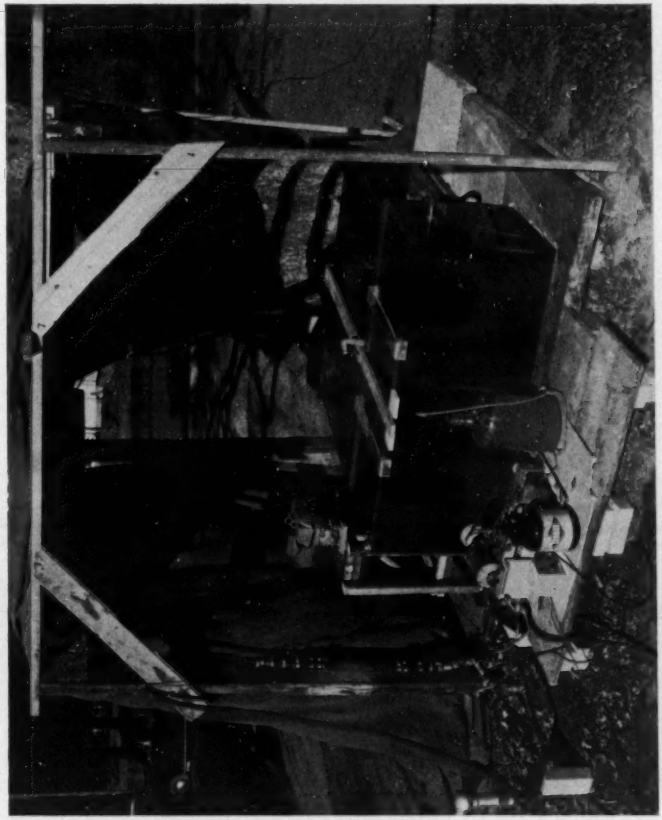


FIG. 8—EQUIPMENT OPERATION

FIG. 9—EQUIPMENT OPERATION

Coagulant Studies

A number of different coagulants were tried to determine the chemicals best suited to this water. The coagulant systems studied were as follows:

1. Alum with activated silica
2. Alum with pH adjustment
3. Ferric sulphate
4. Ferrous sulphate with prechlorination
5. Alum with bentonite clay
6. Alum
7. Alum with prechlorination

All chemicals were added and mixed ahead of the flocculator.

The use of iron salts was quickly eliminated since the effluent was more highly colored than the raw water.

Alum in the various combinations listed formed poor to good floc. Except for combination No. 7, however, the floc did not settle well even when it appeared to be dense and of sufficient size. A series of microscopic examinations was conducted to determine the cause for poor settling. Temperature rise through the unit was only one to two degrees so that the difficulty could not be explained by the buoyancy effect of gas leaving the solution. Microscopic examination revealed that algae activity was heavy with many filamentous forms which prevented the floc from settling. The presence of algae interferes with floc settling since they are generally of the same density as water. Carter (1) has reported on the difficulty of floc settling in low turbidity waters in which algae are present. He found that in the presence of Oscillatoria, which tend to float the floc, a dose of 80 mg/l alum with 40 mg/l lime to form a calcium carbonate precipitate to weight the floc, and 5 mg/l chlorine to kill the algae were necessary in order to obtain good floc removals.

It was found that prechlorination in conjunction with alum destroyed the filamentous forms and improved color removal. Significantly, Daphnia were also affected so that they settled out in the sedimentation tank. This is important in considering a process which would not include filtration, because Daphnia are not normally removed by coagulation and sedimentation, and they are visible to the naked eye after color has been removed.

Accordingly, the selected treatment consisted of alum with

prechlorination. Chlorine was added as sodium hypochlorite. It was necessary to maintain a minimum chlorine residual (OTA) in the sedimentation tank of 2 to 3 ppm to attain adequate color removal. The sodium hypochlorite incidentally acts to bring the pH in the flocculator to within the range of 5.0 to 6.0 which proved optimum for coagulation of this water.

Optimum color removal was obtained with 35 mg/l of alum and a chlorine dose of about 5 ppm.

The resulting sedimentation tank effluent was low in color (15 or less), free of algae and plankton and the MPN coliform count was below 5 per 100 ml. Residual aluminum was less than 1.0 ppm.

Cold-Weather Difficulties

During the first week of December, water temperature in the pond dropped sharply from 45° F. to 37° F. The resulting increase in water viscosity interfered with settling, and an increasing amount of fine floc particles passed through the settling basin.

Attempts to improve operation by adjusting the chemical dosage proved futile. Finally, additional flocculation was provided by installing another flocculating unit designated as the "primary flocculator" ahead of the original unit. The primary flocculator, of 30 gallon capacity, was made from a steel drum. Stirring was accomplished by a laboratory type stirrer using a peripheral paddle speed of 100 fpm.

This unit improved operation so that the color of the sedimentation tank effluent was again satisfactory (20-25) with a residual aluminum of 1 ppm.

The color of the sedimentation effluent was used as the criterion of efficiency in all of the pilot plant runs. Complete analyses were made after good color removal had been maintained for several days.

The analytical results are presented in Table 1.

Summary and Conclusions

A design is presented for a flexible pilot plant for water treatment comprising flocculation, sedimentation, and sand filtration.

The plant was operated for an extended period with highly

colored surface water from Middleton Pond. Special attention was directed to results attained *without filtration* by coagulation and sedimentation only. Floc particles remaining in samples of the sedimentation tank effluent were dissolved by pH adjustment with lime. The data indicate that good color removal was obtained without filtration and that the effluents were of good bacteriological and biological quality, and low in alum.

TABLE 1.—ANALYTICAL RESULTS—PILOT-PLANT STUDY AT MIDDLETON POND
(Alum dosage—35 mg/l)

Chemical Analyses

	Raw water	Settled* water	Filtered* water
pH	6.8	8.5	8.5
Color	65	22	8
Turbidity	5	6	2
Hardness (ppm)	15	25	25
Alkalinity (ppm)	16	22	22
Iron (ppm)	0.10	0.15	0.08
Aluminum (ppm)	0	0.8	0.4
Residual Cl (ppm)	0	2.0	2.0

Coliform Organisms (MPN)

Raw	Settled
50	Less than 3.6

Microscopic Examination

Raw	Settled
590 ASU	0 ASU
400 amorphous	150 amorphous

* pH adjusted to 8.5.

On this basis, a prototype design may be developed for treatment of highly colored waters without filtration, greatly reducing the required capital outlay. Such plants would require closer supervision than conventional rapid sand filter plants and should be limited to those surface waters obtained from clean watersheds.

Certain pertinent observations are summarized below:

1. At temperatures above 45° F., and raw water color of 60, an effluent was produced with color of less than 20 by treatment

with alum at 35 mg/l, prechlorination at approximately 5 mg/l, flocculation for 60 minutes and sedimentation for 75 minutes. This would correspond to a prototype detention time on the order of 4½ hours.

2. At temperature below 45° F., it was necessary to provide auxiliary flocculation in a unit giving an additional 15 minutes of stirring with high paddle velocity. A similar benefit could have been obtained by a longer sedimentation period.

3. High concentration of filamentous algae interfere with sedimentation. They may be controlled by prechlorination.

4. Prechlorination can be beneficial in improving color removal with alum floc.

5. Prechlorination can control free-swimming crustacea which are normally removed only by filtration.

Acknowledgment

Valuable assistance and suggestions were given by Messrs. Worthen H. Taylor, Joseph A. McCarthy, Ralph M. Soule, and Barnet Rosenthal of the Division of Sanitary Engineering, Massachusetts Department of Public Health, and by Vernon C. Russell and Joseph Karonis of the town of Danvers.

Special acknowledgment should be extended to Professor Rolf Eliassen, Department of Sanitary Engineering, Massachusetts Institute of Technology, who acted as advisor to the senior author in the preparation of his thesis on this subject.

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VACUUM-TYPE DIATOMITE FILTERS IN WATER TREATMENT

BY HENRY N. ARMBRUST*

[*Read September 23, 1959.*]

During the past several years the use of the diatomite filter in swimming-pool water purification has increased phenomenally. In 1958 approximately 10,000 municipal and institutional pools were built and 53% of these employed this type of filtration.

While it is true that the problems in pool filtration are different from those in potable water filtration, the fundamental problem is the same and the operating experience of these installations warrants scrutiny by the water-works industry.

A number of installations have been made in the past few years on potable and industrial water supplies where the problem was unusual in that a higher degree of impurity removal has been required than is commonly encountered in municipal water treatment and where these filters have worked out to very good advantage.

In Operation Deep Freeze in the Antractic, nine small filter plants were used for the filtration of melted snow to remove soot from oil fired heaters and penguin guano pollution. Three more plants were shipped in August of this year. As the problem of supply is very serious, it is obvious that the equipment must be dependable and efficient.

Recently a paper company found that it was necessary to use a higher quality water for the manufacture of photographic paper than could be produced by their new sand filtration plant. Not only must the water be of high clarity, but it must be free of radio-active particles. Tests using a gravity-vacuum filter showed that a completely satisfactory water could be produced and a full-scale plant will shortly be installed. The raw water is very highly turbid and comes from a river at a point below another large paper mill which discharges waste sufficient to cause the water to appear like grayish milk. Treatment ahead of the

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filter consists of coagulation and pH correction with aluminum sulphate and sodium hydroxide, chlorination and sedimentation.

At Messina, N. Y., a gravity-vacuum diatomite filter plant is supplying the potable water for the new Reynolds Metals plant. According to a recent report from Reynolds, the operation of the equipment has been very satisfactory and has of course met with approval from the New York State Health Department. The water comes from a pressure main from the St. Lawrence River and there is no pretreatment ahead of the filters.

Winterport, Me., has had a vacuum diatomite filter in operation since early last spring. Here the problem is that of turbidity removal. While there is some color present, it is within allowable limits. The water is taken from a reservoir fed by a stream. At times the turbidity has been in excess of 100 ppm.

At Jamestown, R. I., an experimental 200-gpm plant employing a leaf-type vacuum filter was operated intermittently for a period of about a year on both coagulated and settled water, and water which had received prechlorination and chlorine dioxide treatment only. Results indicated that for most of the year a very satisfactory water could be economically produced without coagulation. During the periods of high color, which sometimes exceeded 100 ppm, it would be necessary to resort to coagulation in order to reduce it to within limits. The overall cost of treatment would be approximately the same as when using the existing old, but satisfactory sand filter. If a new filter had been required it would have been more economical to go to diatomite.

Another interesting installation was made last year in South America by the Aluminum Corporation of America. The only available water supply is a river, the water of which showed a concentration of from 200 to 4000 coliform bacteria per milliliter. Here the natives will not drink chlorinated water, believing it will make them sterile. If the water is chlorinated, they will drink from mudholes or the river itself. Alcoa was told that while the vacuum diatomite filter was quite efficient in the removal of bacteria, it should not be depended upon alone, but should be used with chlorination. Without chlorination as a safeguard, too much depended upon proper operation. In spite of this, equipment was purchased and installed. According to

reports from Alcoa, the bacteria count has been reduced by filtration alone to acceptable limits. This installation is being watched carefully by the local health officials, who are quite enthusiastic about it.

What then are the advantages of gravity-vacuum filter aid filtration, and what are its limitations?

Advantages:

1. A higher degree of clarity in the effluent.
2. Ability to remove suspended solids with or without coagulation.
3. A high degree of bacteria removal.
4. Excellent removal of algae.

Limits:

1. While it will remove "Apparent" color, the removal of true color without coagulation is very low.
2. Alum floc carry-over will blind off the filter cake rapidly, causing short runs.
3. Although in many cases turbidities as high as 60 to 100 gpm have been removed economically without coagulation, there have also been cases where even with relatively low turbidities, the filter runs have been short and labor costs consequently high.
4. Iron floc reacts like alum floc and will shorten filter runs if allowed to be carried into the filter.

Filter Construction

Many of the same fundamental principles pertaining to a sand filter pertain to the diatomite filter. Until recently, all diatomite or filter aid filters were of the pressure type where the filtering surfaces were enclosed in a pressure vessel. Although these were equipped with observation windows or ports, it was impossible to *know* that the precoat was properly applied so that all water would pass through the diatomite cake and be filtered.

Similarly, when the filter was back-washed, it was impossible to know that the filter was entirely clean. As a result, the filter elements became fouled to the extent that they would not properly precoat. Without precoat on a portion of the element to

protect it, more and more of the element became fouled and unfiltered water passed through these areas until a filtering mat of a mixture of filter aid and debris had been built up.

The vacuum filter is an open tank filter with a multiplicity of porous filter leaves or tubes within the tank, connecting to the effluent. Where there is sufficient fall between the filter and the clear well it can be used entirely as a gravity filter. Most commonly, however, the effluent is connected to the suction of a centrifugal pump rated for the desired capacity at a total dynamic head to deliver the flow against whatever back pressure there may be plus the maximum suction which will be developed at the end of the run as the cake becomes clogged. This is usually from 17 to 22 feet.

Because of the open construction it is usually possible to observe the filter cake throughout the entire run. When the end of the run is reached, the influent and effluent valves are closed, the tank drained and the spent cake rinsed or washed from the elements with a stream from a hose or by the use of spray nozzles. This method of cleaning has been found to be more efficient than back washing by means of a reverse flow. Because of this and because any failure of any portion of the elements to take the precoat can be readily observed and the condition remedied, the gravity vacuum filter can be and usually is maintained in a much more efficient condition than the pressure filter. Bauman (1) says—

"The vacuum filter is much simpler to operate than the pressure. The observation of the septums in a vacuum filter during all phases of operation is a definite asset. Any failure in precoating of the septums, or a failure of the septums, sleeves or filter cake, during operation of this filter would be visible to the operator. This is a valuable characteristic in the filtration of a municipal water supply. Filter cake failure in a municipal plant could result in pollution of a potable water supply."

The diatomite filter has been likened to a slow sand filter and like the latter, operates to best advantage at low rates. The usual design rate of such filters when handling water for potable or industrial use is between one half and one gpm per square foot of filter area.

Sizes

The pressure diatomite filter is limited by practicality as to size. As the filter becomes larger, the problem of closure becomes greater and more expensive. In some cases, the closure of a pressure filter shell is more costly than the rest of the filter shell. Where the tank is open to the atmosphere, it need be adequately strong to support the head of water in the tank and this is usually less then six feet.

Under such conditions, it is possible to build tanks of fiberglass reinforced plastic with supporting steel frame works. These can be built in almost any size. At the present time, this type of filter is built for water works use in sizes ranging from 108 sq. ft. to 648 sq. ft. or in other words, for flows of from up to 108 gpm to 648 gpm in one unit.

There is no reason why larger F.R.P. filters cannot be built and they are corrosion-proof.

Several manufacturers are selling filter elements for installation in concrete tanks. Although as far as filtration is concerned, these will operate well, the concrete tank construction renders it inadvisable to acid clean the elements without removing them from the tank, as can be done in the F.R.P. Filters.

Filter Elements

Filter elements are made in a number of forms. While tubular elements have been most common, leaf type elements are in general much more desirable as they offer more plane surface making them easier to wash and to inspect. The element consists of an open core of fabricated metal or plastic which serves as the underdrain and is connected to a means of egress to the filtered water manifold. The core is covered by metallic wire cloth perforated sheet metal or by plastic cloth such as Orlon, Dacron, Nylon or polyethylene. The selection of such cloths is highly important, both as to the type of strand and the weave. Where the entire element is of plastic, corrosion difficulties are eliminated. Bell (2) says of metal elements:

"Electrolytic failures are frequent. These are often due to deposition of particles of iron oxide, from the other filter components or piping, in the septum openings. This is followed by

galvanic action in which the septum furnishes the sacrificial metals. Other failures result from galvanic action between elements and other filter components, which can be lessened by use of insulating connections between component parts of filters."

Basic Equipment Required

1. A level control to maintain proper level of water over the filter elements.
2. Filter.
3. Vacuum gauge.
4. Filter pump.
5. Vacuum limit switch.
6. Equipment for body feed. (Dry feeder with slurrying flume)
7. Rate of flow indicator.

Desirable Additional Equipment

1. Rate of flow controller.
2. Chemical Feeders.
3. Chemical feeder and diatomite feeder controller.
4. Recording meters for flow and vacuum.
5. Automatic mechanically or hydraulically operated valves.
6. Automatic washing equipment.

The level controller should be a tight shutoff valve, so arranged as to serve as the influent shutoff during washing or when the filter is taken out of service for any reason.

The filter should be capable of operating at ½ gpm per sq ft for normal demand and not more than 1 gpm per sq ft for normal peak demands.

The filter pump should be an end suction pump, either close-coupled or driven through a flexible coupling. The gland or seal should be under pressure.

As pointed out by Bell (3), the body feeder should be of the dry type with a slurrying flume and eductor so that the filter aid can be introduced into the influent line for good mixing.

The rate of flow indicator should be any one of several types but preferably one which can be used in conjunction with a rate controller and chemical feeder control as well as a recording meter.

The rate-of-flow controller may be either a standard water works unit or some other type which will interpose very little pressure loss when nearly wide open.

Bauman (1) and Bell (3) concluded that diatomite filtration of water is economical in comparison to sand filtration. Bauman ran extensive tests on iron-bearing waters, using both pressure and vacuum filters, during the period of September 1, 1958, and May 31, 1959, under a research grant from the National Institute of Health, U. S. Public Health Service, to Iowa State University. Bell's work was with a conventional sand filtration plant in which a battery of pressure diatomite filters had been installed and which could be used interchangeably with the sand filter. This extended over a period of years, using Raritan river water as the source. The water varied widely in turbidity and algae content. Turbidity ranged from 2 - 800 units (by silica scale determination), averaging 11 units. Color ranged from 3 - 400 ppm averaging 28. Pretreatment was accomplished in an upflow basin, the operation of which was upset at times due to varying temperatures and the flashy chemical characteristics of the water.

Bell's work was particularly valuable as it presented cost comparisons between the sand filter operation and diatomite operation on both pretreated and untreated water. His figures for the installed cost of 2 mgd water plants as of 1956 were:

Diatomite filtration of raw water	$161,630.00
Diatomite filtration of pretreated water	$201,790.00
Sand filtration of pretreated water	$296,581.00

Based on an annual production of 560 m.g., the total operating cost, including chemicals, power, maintenance, amortization and debt service was as follows:

Diatomite filtration of raw water	$45.71/m.g.
Diatomite filtration of pretreated water	$55.22/m.g.
Sand filtration of pretreated water	$45.31

As the pressure filter requires a pump designed for a total dynamic head of 100 ft., whereas the vacuum filter requires only 35 to 45 ft. T.D.H., the power costs for the latter would be lower. On the other hand, it is possible that because the pres-

sure filter can be operated to higher differentials, the cost of diatomite might be greater for the vacuum filters. This, however, would tend to be equalized by the ability to maintain the filter septums of the vacuum filter in a cleaner average condition.

It is felt that consulting engineers are in many cases doing their clients an injustice if they do not investigate the applicability. Similarly, Health Department Engineers should permit pilot-plant operation or give provisional approval to installations which are nearer full plant size—for instance, one filter of a proposed battery. This is by far the best, as the operators gain experience in handling an actual component of the proposed plant.

It is generally agreed by those interested in the development of diatomaceous-earth filtration that a proposed diatomite filter-plant project should be handled under the guidance of a competent consulting engineer, that full chemical and physical data should be obtained on the water and, if possible, a series of pilot runs should be made on the water during the season's high turbidity and high algae (1, 3, 4).

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IRRIGATION-WATER QUALITY OF THE NEW YORK-NEW ENGLAND REGION

BY J. A. CREATURA*

[*Received October 14, 1959.*]

ABSTRACT

A sharply rising curve exemplifies the use of water for irrigation in the northeast. The quality of New York and New England water resources is generally satisfactory for irrigation. Salinity, sodium and residual sodium carbonate hazards are not serious impediments to irrigation. Northeastern waters seldom have toxic elements naturally present but in some instances they may be added through pollution. The increased use of water for irrigation will result in deterioration of the quality of the water resources.

INTRODUCTION

Agriculture, our most vital industry, uses tremendous volumes of water for irrigation. Kohler (2) estimates that between 75 and 100 billion gallons of water are used each day to irrigate America's crops. Seventeen western states, collectively termed "dry" because they ordinarily experience low precipitation, are the biggest users. Approximately 90 per cent of the nation's irrigated land is located in these states.

Although the irrigated land areas of the humid eastern states are small in comparison with those of the west, the east is experiencing a greater rate of expansion. From 1949 to 1954, the number of acres of irrigated land in the west increased from 25,248,682 to 29,215,869 acres; an increase of 16 percent. During the same period, irrigated land in the east increased 75 percent, from 1,539,259 to 2,693,011 acres. The New York-New England region showed an increase of 92 percent between 1949 and 1954; 50,698 acres in 1949 to 97,424 irrigated acres in 1954. The eastern farmer, according to these figures, is rapidly learning the value of irrigation.

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Eastern and western irrigation practices differ in certain respects. During the growing season, irrigation water is applied heavily to western crops while eastern crops require only supplemental application during critical periods of the growing season. Therefore, for reasons of quantity, the danger of crop and soil damage resulting from using water of poor quality does not appear to be as serious in the east as in the west. Furthermore, eastern total annual precipitation is heavy, though not always well distributed. Heavy precipitation may alleviate the east's irrigation water problems by diluting and leaching away harmful concentrations of dissolved materials deposited by irrigation waters. Nevertheless, the possibility of damage to eastern crops and soils does exist and the quality aspects of irrigation water require consideration.

Quality Considerations

A number of agencies are currently investigating irrigation problems—notably the U. S. Salinity Laboratory of the Department of Agriculture. Although primarily concerned with western irrigation problems, many of their recommendations may be applied, with some qualification, to eastern agriculture.

The Salinity Laboratory (3) suggests a number of quality criteria for classifying sources of irrigation water. They are:

1. Salinity hazard
2. Alkali hazard
3. Residual sodium carbonate
4. Toxic elements

In actual practice the irrigator must recognize that management factors also have an important role since a properly managed irrigation system often allows the use of water of doubtful quality.

Salinity Hazard

An excessive concentration of dissolved salts may prohibit the use of a source of water for irrigation. The degree of salinity hazard encountered in irrigation waters is directly proportional to the total salt concentration. Specific electric conductance is a good approximation of the salt content of a water.

The relationship between the degree of salinity and specific

conductances (3), as proposed by the Salinity Laboratory, is shown in Table 1.

TABLE 1.—THE SALINITY-HAZARD CLASSES OF IRRIGATION WATERS

Degree of salinity hazard	Low	Medium	High	Very high
Specific conductance range in micromhos	0–250	250–750	750–2250	in excess of 2250

The salinity hazard of eastern waters used for irrigation has not been investigated fully. However, it has been reported (5) that, in general, assuming the root zone is leached by rainfall at least once a year and probably more often, high salinity hazard waters (750-2250 micromhos) may be used supplementally with little danger to crops and very high salinity hazard waters (in excess of 2250 micromhos) can be used occasionally on all but the most salt sensitive crops. Extremely high salinity hazard waters with conductances three or more times in excess of 2250 micromhos are of doubtful utility and probably should not be used for supplemental irrigation.

An examination of ground water quality data indicates that low and medium salinity classes are probably typical of New York State ground-water sources. As Table 2 illustrates, 71

TABLE 2.—SALINITY-HAZARD CLASSIFICATION OF NEW YORK GROUND WATERS

Salinity-hazard class	Number of sources	Percent of total
Low (0–250 micromhos)	103	25
Medium (250–750 micromhos)	192	46
High (750–2250 micromhos)	72	17
Very high (in excess of 2250 micromhos)	48	12
	415	100

percent of the ground waters, examined are in the low and medium salinity hazard class. However, examples of waters with specific conductances in excess of 2250 micromhos are not infrequent; most of them occurring in areas along the upper St. Lawrence River, west-central New York and Long Island.

The surface waters of New York State are generally in the low salinity hazard class—specific conductance range of 0-250

micromhos. In the west-central portion of the state, however, some surface waters lie in the 750-2250 micromho range; the result of the influence of the Salina formation of that region. Rarely do specific conductance measurements exceed 2250 micromhos—the upper limit of the high salinity class.

Information on the quality of New England ground waters is scanty. Based on available data, New England ground waters present low salinity hazards (Table 3). Further study may uncover ground-water sources that are highly saline, especially in coastal areas.

TABLE 3.—SALINITY-HAZARD CLASSIFICATION OF NEW ENGLAND GROUND WATERS

Salinity-hazard class	Number of sources	Percent of total
Low (0–250 micromhos)	169	75
Medium (250–750 micromhos)	54	24
High (750–2250 micromhos)	3	1
Very high (in excess of 2250 micromhos)	0	0
	226	100

Surface waters in New England generally lie in the low salinity-hazard class. Exceptions are found in locations relatively near the coast where salt water encroachment is a quality factor.

Based on the salinity hazard alone, the water resources of New York and New England may be regarded as generally suitable for supplemental irrigation.

ALKALI HAZARD

Salinity-hazard considerations alone do not define the quality of irrigation waters. Equally important is the degree of alkali or sodium hazard an irrigation water presents to a soil.

To predict the presence of an alkali hazard, use is made of an expression called the sodium adsorption ratio (SAR). SAR values are calculated as follows:

$$SAR = \frac{Na^{+}}{\sqrt{\frac{Ca^{++} + Mg^{++}}{2}}}$$

The concentrations above are expressed as milliequivalents per liter. High proportions of sodium to calcium and magnesium

concentrations represent high SAR values and correspondingly high alkali hazards. Conversely, low sodium: calcium-magnesium ratios represent low SAR values and low alkali hazards.

The SAR values of ground waters in New York are generally found to be less than 10. Since SAR values of this magnitude do not prohibit the use of a water in the west, the same rule can apply to eastern waters.

In New York, SAR values of surface waters are generally lower than those of ground waters. Consequently, the alkali hazard encountered in the use of these surface waters is slight.

With the exception of a few isolated examples along the coast, ground and surface water resources in New England generally have very low SAR values.

Wherever relatively high SAR values have been determined in New York and New England, leaching and dilution during periods of heavy rainfall would tend to minimize the alkali hazard.

Residual Sodium Carbonate

Closely related to the sodium problem is the difficulty brought about by disproportionately high concentrations of bicarbonate in an irrigation water. To predict the presence of a bicarbonate problem in an irrigation water, reference is made to the magnitude of its residual sodium carbonate (RSC) value. Residual sodium carbonate, the carbonate in excess of calcium and magnesium free to form the highly soluble sodium carbonate, is calculated in the following manner:

$$RSC = (CO_3^{=} + HCO_3^{-}) - (Ca^{++} + Mg^{++}).$$

All concentrations are expressed in milliequivalents per liter.

The U. S. Salinity Laboratory (5) postulates that 2.5 or more milliequivalents per liter of residual sodium carbonate renders a water unsuitable for irrigation purposes; residual sodium carbonate between 1.25 and 2.5 milliequivalents per liter are marginal, and concentrations of less than 1.25 milliequivalents per liter are probably safe.

Ground waters in New York generally present no residual sodium carbonate problems. Occasionally, however, residual sodium carbonate values exceed the upper limit of permissible tolerance recommended by the Salinity Laboratory.

Analyses of New York State surface waters reveal no example where residual sodium carbonates exceed 2.5 milliequivalents per liter. Similarly, for both ground and surface waters of New England, no example can be found of dangerous residual sodium carbonate concentrations.

The residual sodium carbonate concentrations of northeastern waters are low; acceptable by western standards and certainly not prohibitive for supplemental irrigation purposes in the east. Some ground waters in New York are exceptions. However, most waters could certainly be used for relatively short periods if heavy rains are spaced sufficiently close together to permit leaching.

Toxic Elements

The final consideration necessary to establish the utility of an irrigation water is the determination of the presence or absence of concentrations of elements directly toxic to crops. There are many elements known to be toxic. Some are essential to crop nutrition but cannot be present in the nutrient solution far above optimum concentrations; others are not essential to plants though absorbed and accumulated in plant tissue.

Boron is an essential plant nutrient required in trace amounts. Nutrient solution concentrations of boron slightly in excess of the amount needed for optimum growth are toxic. For irrigation water, Bear (1) suggests that 0.66 ppm be the upper limit for boron sensitive crops, 1.33 ppm for semitolerant crops and 2 ppm for tolerant crops.

In some areas of the west, boron is present in the soil as soluble sodium borate which is easily leached into irrigation water sources. As a result of this, the boron concentrations of western irrigation waters are often dangerously high. Eastern boron deposits ranging from 25 to 100 ppm in soils are found in the form of the highly insoluble mineral tourmaline. Because of the relatively insoluble boron deposits in the east and the noted boron deficiency in some areas, it is safe to assume that boron is not present in harmful concentrations in northeastern waters. Tables 4 and 5 represent random samplings and analyses for boron.

The boron concentrations listed in Table 3 and 4 are low but for two exceptions, both ground waters in Hampden County, Massachusetts. Although these two ground waters contain com-

paratively high boron concentrations, their use as irrigation water sources is not prohibited if a crop semi-tolerant to boron is planted. For supplemental irrigation these two ground waters could probably be used on any crop.

TABLE 4.—BORON CONCENTRATIONS OF SOME NORTHEASTERN GROUND WATERS

Location of well	Depth (ft)	Boron (ppm)
Newburgh, Orange Co., N.Y.	303	.13
Saratoga Springs, Saratoga Co., N.Y.	468	.20
11 mi. North of Machias, Washington Co., Me.	170	.00
Holyoke, Hampden Co., Mass.	404	1.4
Springfield, Hampden Co., Mass.	525	1.3
S. Hadley, Hampshire Co., Mass.	12	.05
Limestone, Aroostook Co., Me.	224	.04
20 mi. N.E. Concord, Essex Co., Vt.	750	.00
Near Sanborn, Niagara Co., N.Y.	40	.08

TABLE 5.—BORON CONCENTRATIONS OF SOME NORTHEASTERN SURFACE WATERS

Location	Boron (ppm)
Neversink River at Godeffroy, Orange Co., N.Y.	.00
Seneca River at Baldwinsville, Onondaga Co., N.Y.	.03
Delaware River at Port Jervis, Orange Co., N.Y.	.00
Rutgers Creek at Gardnerville, Orange Co., N.Y.	.03
Naugatuck River at Thomaston, Litchfield Co., Conn.	.00
Quinebaug River near Putnam, Windham Co., Conn.	.00
Farmington River at Rainbow, Hartford Co., Conn.	.00
Charles River at Watertown, Middlesex Co., Mass.	.05

There is some doubt as to the nutritive value of sodium. It is accumulated by plants, but its physiological function, if any, is not clear. In any case, excessive accumulations of sodium do cause plant damage, especially in the sodium sensitive stone fruits. For most crops, however, the salinity and alkali hazard aspects of irrigation water would far outweigh any consideration of the toxic character of sodium. Furthermore, the east's heavy rainfall would minimize the danger of sodium toxicity by leaching sodium salts out of the root zone.

Some plants, when exposed to high concentrations of non-essential chloride in the nutrient solution, suffer toxic damage. Peaches, pecans, grapes and tobacco are some of the chloride sensitive crops. Whitt and Van Bavel (4) report that chloride in excess of 25 ppm in an irrigation water reduces the quality of tobacco. Chloride content is frequently quite high in northeastern waters, and caution should be exercised in the selection of an irrigation source to insure the health of a particular crop. Again, the heavy rainfall of the east would aid in reducing the chloride content of the nutrient solution.

There are many other elements potentially toxic to plants. Among them are arsenic, fluorine, copper, aluminum, barium, lithium, nickel, zinc and lead. It is conceivable that such elements may be discharged into water resources in concentrations dangerous to crops. Whether or not concentrations of these elements remain soluble or precipitate as insoluble salts is dependent upon several factors. Ordinarily, these elements are not found in natural northeastern waters in toxic concentrations.

Future Quality Problems

The quality of irrigation water sources may change with use. This is possible when heavy withdrawals are made over relatively long periods of time. Should this condition arise certain problems may become apparent.

In ground-water supplies, water tables will be lowered, and, if recovery is insufficient, the yield may be seriously affected. This appears to be only a quantity problem. However, water of poorer quality will be pumped if the lowering of the water table induces water of poorer quality laterally and with depth. For example, lowering of the water table in wells bordering streams may result in infiltration of surface water. If the surface water is highly mineralized, either naturally or because of pollution, the quality of the ground water in turn would be adversely affected.

Heavy withdrawals from surface waters, especially small streams, will lower the rate of discharge downstream. This situation may raise a legal issue of riparian rights. Technically, however, reduction in flow will also reduce the dilution capability of a stream so that the quality of water downstream may become poorer.

The problems discussed are certainly possible. Time will tell whether they come into being, and their magnitude. In the meantime, following the old proverb, "A stitch in time saves nine," periodic checks on the quality of irrigation waters are recommended.

Summary

The chemical quality of New York and New England's water resources is generally satisfactory for supplemental irrigation purposes. Salinity and sodium problems are not serious impediments to irrigation for most areas of the northeast. Few examples of high residual sodium carbonate have been found for waters of the northeast. The addition of toxic elements to a soil by irrigation water is possible in isolated instances because of pollution, but concentrations of most toxic elements are not usually present in natural waters. In contrast to the west, the heavier rainfall in the east, through the leaching process, reduces the salinity, sodium and residual sodium carbonate hazards. Although water resources in the northeast are generally suitable for irrigation, exceptions do occur. The potential irrigator should be aware of the existence of these exceptions and their attendant dangers.

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FINANCING WATER-WORKS CONSTRUCTION IN NEW ENGLAND

BY WILLIAM P. COUES*

[*Read November 19, 1959.*]

MEMBERS AND GUESTS—NEW ENGLAND WATER WORKS ASSOCIATION:

I am very glad to be here today and honored that you are willing to put up with me as a speaker! No doubt you'll regret it in a few seconds when you will readily see that you know a lot more about my subject, "Financing Water Works in New England", than I do. Just so there isn't any doubt about that, I want you to realize that it is I who bring it up now!

Your business and careers are interesting ones in that you deal in an essential "commodity". Some peculiar people think money is also essential, and so we have "essentiality" in common.

Now, before I get into the adequacies or inadequacies of the do's and don't's department, I would like to review with you the recent financial history of some water districts taken at random and ranging in size from $435,000 to $4,000,000. These are all New England Water Districts.

Here's a district with a total bond issue of $475,000. which was sold in December, 1956 at 3.91%. As an initial borrowing in anticipation of the ultimate bond issue they chose an amount of $85,000. in September, 1955, which we let them have at 1¼%. This then was a borrowing of approximately 20% of the total amount. Eight months later they needed more temporary money amounting to $350,000. which we let them have at 2.40%, interest rates having advanced in the meanwhile. That is a rather dull, uninteresting and simple example of borrowing in anticipation of a bond issue.

Here is another which follows a similar pattern. This is a $4,000,000. project with an initial borrowing of $350,000.—a little less than 10% of the total amount at 2.40%, and an addi-

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tional loan 6 months later of ½ million at a rate of 1.10%, the substantial down in rate due to the easy money conditions prevailing at that time due to the government's combatting the recession. I point out with pride that the rate was down despite the fact that the margin of loan to total project cost was substantially higher. You see, we bankers aren't such Scrooges after all. We're a pretty decent lot when our feathers are not rubbed the wrong way!

One more similar situation varies in degree a little bit and is worthy of mention. This pertains to a $430,000. issue at 3.50%, an initial clear-cut borrowing of an even $100,000. at 2.35% and increased to $250,000. I cite this one because here the temporary borrowing exceeded 50% of the cost of the project. So what we have now seen are temporary borrowings ranging from 20% of the total amount up to in excess of 90% in the first case cited.

Now here is a very different one, $450,000. where the shrewd Yankees borrowed an initial $3,000., then $1,000. more and then they really hit their stride and borrowed every single month for 14 consecutive months in amounts ranging from $17,000. per month to $50,000. This is an expensive and laborious way to do it in many ways, as in each instance a separate legal opinion was essential, costing at least $50. or a total of $700.

Bonds for Town water, Water District water, Sewer District or School bonds require the same financial procedure and your financial consultants can help you outline the procedure.

Proper Valuation to Support Debt

Make sure there is a proper valuation of city, town or district to support a major aggresive and important program.

Example: An expected indebtedness of $350,000. against a valuation of $900,000. is not good. A much better relationship would be $113,000. indebtedness compared to a valuation of $691,000. A poor relationship of this sort might cost at least ½ of 1% extra in coupon rate. In Maine where the Public Utilities Commission likes term bonds instead of serials, we think it costs those Districts ¼ of 1% extra depending on the District.

Your banker will gladly give you an "educated guess" as

to the cost of money and also advice as to when an issue can best be marketed. But remember, this is a changing scene, and conditions existing at the time of the "educated guess" may not prevail later on.

Our current thinking as to the cost of money on bond issues is 4%, assuming an approximate 20-year issue.

Water Districts are not accepted as tip top credits, and this is wrong, but that is the feeling in the market place. Many of them are actually better than direct town credits. There is a pretty ready market for these bonds, however.

Some dealers do not like small issues; *i.e.*, $75-150,000. We consider moderate ones from $200-500,000.

A few pitfalls to be avoided—

1. Remember that this is a specialized field and just as you are absolutely essential experts in the construction end, others are needed. The next most important expert needed is adequate specialized legal advice.

Do not let your consultants think they can save money by using the local lawyer only. No doubt he's a fine fellow, well liked and capable in his field. But, I submit, he is not a specialist in bond financing. Why do we need such a specialist and who are they? We need them because they will probably save money in the long run. Because of their general reputation—like the Tiffany mark on silver—the wholesale buyer of the bonds, the bond dealer—will require you to pay less interest if this important seal of approval, so to speak, is attached. Why? Because he can turn around and sell the bonds more quickly at a profit. The best arrangement is to have the local lawyer work along with bond counsel as a team.

Who are these so called experts, and where are they located?

Boston: Ropes, Gray, Best, Coolidge & Rugg; Storey, Thorndike, Palmer and Dodge.

Portland: Verrill, Dana, Walker, Philbrick & Whitehouse.

Montpelier: Peter Giuliani.

New York: Mitchell, Pershing, Shetterly & Mitchell.

The first two and the last cover all New England. The other two specialize in their states.

Do not delay the choice of the banker to handle the bonds

any longer than necessary. By getting him in early, he will give you—we hope—wise advice and counsel.

Do not overlook the need for adequate factual financial and historical information of the district, town, city or area.

Do not come into the bond market more than once a year.

Do not borrow in anticipation of the bond issue in tiny amounts of $5,000. or so every time you need a little money. It will cost you more in the long run because of the cost of handling and extra paper work. There will be a legal charge for each separate loan, as I cited previously in one of my examples. Borrow in larger chunks of $25,000-$50,000-$100,000. at a time. Excess cash may be invested in U. S. Treasury 90-day bills—paying at the moment about 4-4.25%, and one came out this week at 4.35% (11/17/59). These are highly liquid and readily salable investments.

Do not let the bond issue exceed the life of the improvement and avoid odd amounts in serial maturities.

Do not delay the award when the bids are opened. This should be made or rejected almost immediately.

Do not plan a bond sale for a Monday or a Friday.

Enough of the DO NOTS. Now for a few constructive things.

No matter how good you may think your charter is, have it reviewed early in the game by BOND COUNSEL. This may save a great deal of time and many a headache. A slight legislative change might be necessary, which is very time consuming.

Tell your banker the whole story from beginning to end. How much you'll need and when is important, and you'll find him helpful, sympathetic and understanding. Just because all these dealings are somewhat unfamiliar to you, don't let it cause you embarrassment. He welcomes you at his office for frank discussions of potential problems. If you can't come in, don't hesitate to call on the telephone to get a rough idea of the prevailing cost of money on bond issues.

You may get a little irked at some of the red tape required by the banker, but remember he has to make SOUND LOANS and he must be able to back up his decisions to the Bank Examiners and his Board of Directors. Don't overlook the fact that he is running a demand deposit shop, that the bank may have to raise money to carry on its business by selling some of

your short term notes. In that case, the banker will also have to satisfy the buyer as to the soundness of the loan. This may help you to understand the reasons for some of the red tape.

What kind of bonds are you going to sell? There are several general types such as

1. The General Obligation with unlimited taxing power.
2. The Combined Revenue and General Obligation with unlimited taxing power.
3. The Straight Revenue Bond: (a) Some with default allowing property seizure; (b) Some with no recourse to tax or property seizure.

The details of these are best left to consultation.

What the Bond Dealers Like

1. Even maturities in multiples of $5,000. or $10,000. each.
2. Complete payment of the indebtedness through bond maturity spelled out in issuing circular without the necessity and uncertainty of further financing in the future.
3. Good coverage of earnings over and above maximum principal and interest requirements (about 1-20 times coverage is minimum requirement). As this improves, so does rating and salability.
4. A rating by one or all of the following: Moody's Investors Service, Standard & Poors Corp., Dun and Bradstreet.
5. Good tax collections.
6. Low tax rate.
7. Low debt in relation to assessed valuation and population.
8. An increasing trend of assessed valuations.
9. A declining debt for the last several years.
10. An area of stable industries and stable people.

Public Sale or Private Sale

Sometimes the advantages of one outweigh the other, but in general I am inclined to favor the public sale. In this way there can never be any justifiable criticism as to the lowest bid being accepted. The *Public Sale* is a planned attack by which the broadest market of many dealers and bankers is sought, wherein a specific hour and place are set for the opening of sealed bids.

Private Sale is the acceptance of an offer made by a single individual firm or institution without competition from others.

A LABORATORY STUDY OF SULFAMIC ACID AS A VEHICLE FOR CHLORINE

BY JOSEPH A. McCARTHY*

[*Read January 21, 1960.*]

Introduction

The explosive growth of backyard pools throughout the country is bound to be of growing importance to public health. Most public or semipublic pools have been provided with facilities for disinfection. But very few of the private pools, simple or elaborate, have such facilities. Draining and refilling is often found to be tedious and expensive. In most cases any attempts at sanitation are strictly left to do-it-yourself technics at the hands of a householder with only a vague idea of what to do.

Judging from the many calls we have received, many pool owners are aware to some extent of the sanitary aspects involved, and most of them know a little about chlorine. Some of them consider it necessary to keep a pool reeking with bleach. But others regard chlorine as corrosive and poisonous but so omnipotent that an eye-dropper full a month is enough to keep a pool clean. The average pool owner, and too many other people as well, do not realize that chlorine is not only an effective killer of bacteria, but is also an omnivorous oxidizing agent.

Factors in the Consumption of Chlorine

The factors involved in the dissipation of chlorine are many and varied. The initial requirement of the water itself must of course be satisfied, but once met, this is not a continuing demand. But many other factors, especially in an open reservoir or in a swimming pool, do continue to add their demands and to utilize the chlorine intended expressly for bacterial kill. Vegetation, pollen, fly ash, stray vapors, casual grit from the atmosphere, all sorts of organic and inorganic materials brought in by bathers, all these contribute to this theft (see Fig. 1).

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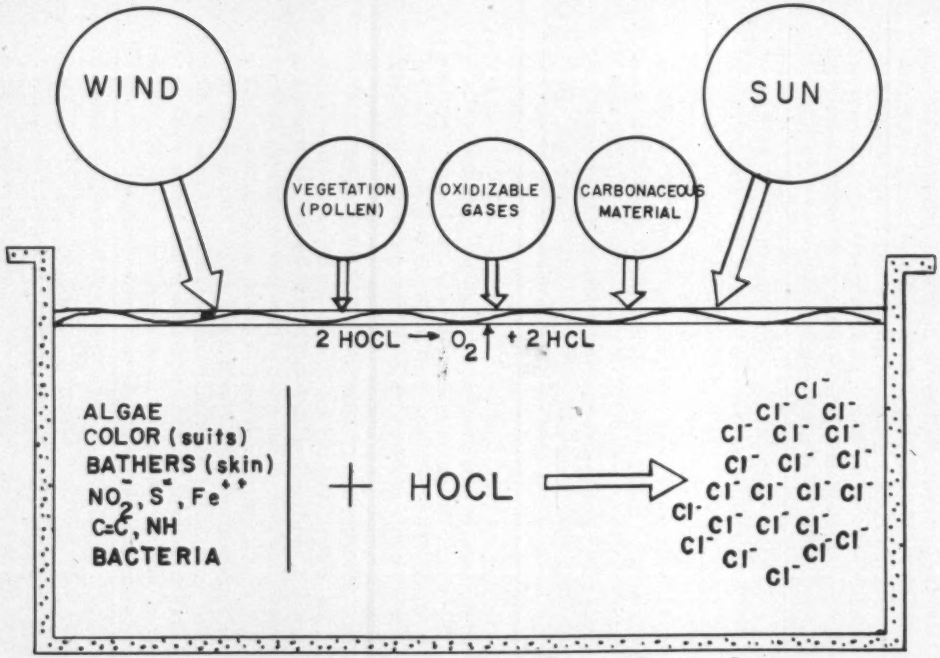

FIG. 1. CONSUMPTION OF CHLORINE KILLING POWER IN SWIMMING POOLS

The very sun and winds rob the pool of active chlorine. The sunlight readily decomposes the hypochlorous acid, and in addition stimulates algal blooms, which make an effective residual impossible to maintain. The winds and the activities of the bathers agitate the water and increase the area of water-air interface through which chlorine may escape.

Preliminary Studies on Stabilizing Chlorine

If these pathways of escape set up by the sun and wind and splashing bathers could be diminished, but more importantly if the oxidizing power of chlorine could be directed more specifically at bacterial kill, a much more effective use of chlorine could be established. In an ideally stabilized system, oxidizing power would be released only to kill bacteria, and when not thus needed would be securely combined to prevent wasting on casual reductants.

We have long been studying possible vehicles for carrying active chlorine. Chlorine dioxide was shown to be moderately satisfying in certain conditions.[1] Ammonia has of course long been advocated, but there seems to be general agreement that the chloramines are relatively slow and ineffective,[2] especially at pH levels desired in swimming pools or in drinking water; and ammonia furnishes food for algae and for many colored bacteria. Organic amino compounds readily combine with chlorine, but they obviously would contribute considerable amounts of potential chlorine demand and food for bacteria and algae. Chloramine T and Halazone, and similar compounds have shown usefulness, but they contain much organic carbon and little chlorine. We became interested in reports of the use of sulfamic acid in breakpoint studies by Savinelli and Cooke.[3] This entirely inorganic acid, NH_2SO_3H, readily combines with chlorine, in dilute concentrations, to produce compounds which appear to be very much like the familiar amines produced by the combination of ammonia and chlorine, as indicated in these reactions:

$NH_3 + Cl^-$	NH_2Cl monochloramine
$NH_2SO_3 + Cl^-$	$NHClSO_3H$ n-chloro-sulfamic acid
$NH_3 + Cl^-$	$NHCl_2$ dichloramine
$NH_2SO_3H + Cl^-$	NCl_2SO_3H n-n-chloro-sulfamic acid

We began preliminary work by adding to buffered water

equivalent amounts of chlorine alone, and chlorine as chloramine, and both of these in combination with sulfamic acid.* To these waters we added at intervals small amounts of filtered settled sewage, and observed the effect on the ortho-tolidin residuals. As shown in Figure 2, practically all of the initial 10 mg/l of chlorine alone was exhausted by three increments of sewage, while the residual of about the same amount given by the combination with sulfamic acid was reduced only about 40%. Chloramine acted in similar fashion. This experiment was repeated with other waters and with varying ratios of chemicals, all with quite similar results. The possibilities of using the sulfamic acid in stabilizing chlorine certainly seemed well worthy of further investigation.

It should be stated that all of the data and observations presented in this paper have been obtained through laboratory scale studies, and consequently their application to actual practice remains to be investigated.

Properties of a Sulfamic Acid-Chlorine System

It was found that the sulfamic acid and chlorine combinations gave reactions similar to those of chlorine alone in liberating iodine from iodides, reduction by thiosulfate, and quantitative reaction with o-tolidin, so that direct measurements and comparisons with chlorine seemed possible. It was a little surprising to find a complete absence of the typical odor of chlorine.

The following questions in regard to the properties pertinent to public health seemed to require answers:

(1) Is sulfamic acid toxic?
(2) Are the mixtures or combinations bactericidal?
(3) For how long is a killing environment maintained by a sulfamic acid-chlorine combination as compared with the same amount of chlorine alone or with ammonia?
(4) Can the presence of an active concentration be measured by a simple test?

1. Toxicity?

The Haskell Laboratory of Industrial Toxicology has made oral and local studies of sulfamic and sulfamates on white rats

*Sulfamic acid (99+%), Eastman Kodak, 4659; sulfamic acid, crystal technical, Dupont (Graselli).

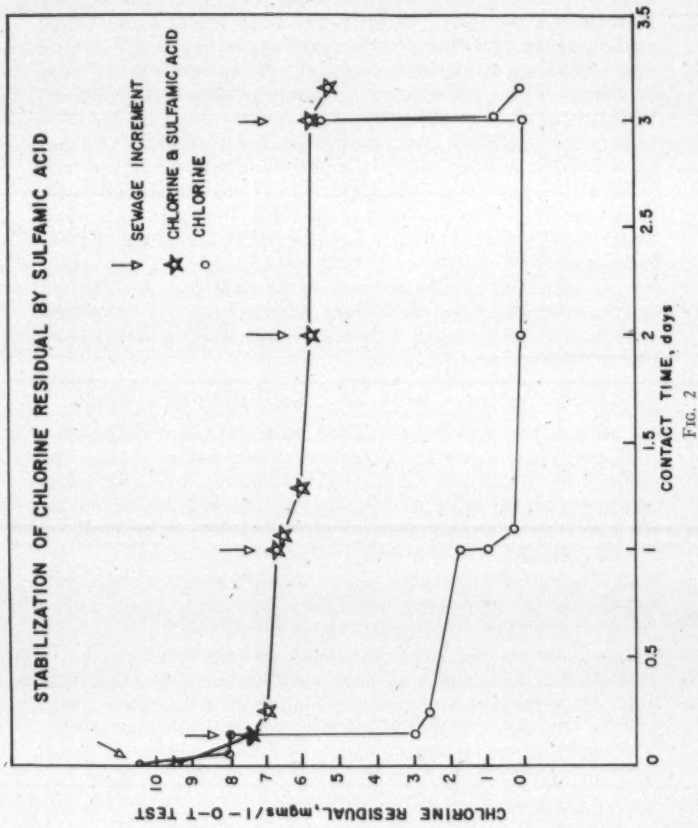
STABILIZATION OF CHLORINE RESIDUAL BY SULFAMIC ACID
SEWAGE INCREMENT
CHLORINE & SULFAMIC ACID
CHLORINE
CHLORINE RESIDUAL, mgms/l - O-T TEST
CONTACT TIME, days
0
0.5
1
1.5
2
2.5
3
3.5
0
1
2
3
4
5
6
7
8
9
10

FIG. 2

and dogs which indicate that the stomach is affected only by concentrations in the order of whole percents, and that intact skin will not be injured by any concentration or by the dry material. Moderate doses had no systemic toxic effects, apart from the irritant action due to the acidity of the chemicals. From these data, it hardly seems likely that a few mg/1 would be toxic.

2. Bactericidal?

In order to provide an intelligent answer to the questions of bactericidal effect of mixtures, it seemed necessary to set up some sort of standard. After some preliminary work it appeared that an acceptable standard might well be a reduction in one hour of 98% of the bacteria grown on nutrient agar at 35° C in 24 hours. Coliform determination were made on enough samples to indicate that with a 98% plate kill the MPN of coliform would not be over 230 per 100 ml, and generally much less, often with coliform confirming only in a few 10 ml portions. We felt that this was low enough to meet any realistic pool standard, and also felt that one hour was not too long to allow for disinfection of a pool.

Our first studies were more or less qualitative, but they indicated that sulfamic acid-chlorine mixtures could kill coliforms and other bacteria in pure culture suspensions, in water and in dilute sewage. They also showed that the mixtures were at least as potent as equivalent amounts of chlorine.

3. Efficiency and Duration of Bactericidal Power

To test continuation of killing power, we set up what might be accepted as a reasonable simulation of pool conditions when the water was not changed daily, by adding to our test waters, at appropriate intervals, small amounts of sewage, which would provide not only the bacteria but the heterogeneous organic matter which might be expected to enter a pool.

In each of our experiments suitable amounts of various types of waters were dosed according to estimated chlorine requirements with chlorine, or chloramine and with mixtures of these with sulfamic acid. It was usually necessary to adjust pH, because of the acidity of both chlorine and sulfamic acid. O-tolidin residuals were read. After some predetermined interval

the desired amount of settled sewage, filtered through paper, was added. After 15 and 60 minutes, and again at other intervals in certain runs, residuals were again determined and at the same time samples were taken out into bottles containing thiosulfate from which plates and coliform tests were set up.

After another interval, generally 24 hours, another increment of sewage was added, and the procedure above was repeated. This was continued day by day until the chlorine was exhausted or until plate counts showed a kill of less than 98%. In a few runs, rechlorination was used. This has not been thoroughly explored, however.

In our earliest quantitative work, on water with low demand, with doses of from 0.44 to 1.6 mg/1, the presence of sulfamic acid made no difference; this was equally true with high doses of the acid, 20 mg/1, or low, 2 mg/1. During these runs we noticed for the first time that killing power had been exhausted in mixtures which still had what might be considered as effective residuals. In these and succeeding runs it became apparent that as the ratio of chlorine to acid was equal to or greater than that required for stoichiometric production of mono-chlorine compound, the mixtures began to show definitely better performances than the chlorine alone. It also began to appear that stability would require doses of acid of perhaps 10 mg/1, regardless of the amount of chlorine.

Of the experiments based on the early indications, three have been chosen for detailed presentation. The first run to be described was made on a water relatively high in inorganic solids and alkalinity, but with a chlorine requirement of only 1.2 mg/1 in 24 hours.

Portions of this water were dosed as follows: A, 20 mg/1 of sulfamic acid followed by 10 chlorine; C, 20 sulfamic and 20 chlorine; B, 10 of chlorine only, D, 20 chlorine only, as shown in Table 1. B, 10 chlorine only, gave a complete kill of plate bacteria in 15 minutes with the first three increments, leaving a residual of 0.30, then dropped to 88% on the fourth and none on the fifth. Portion A also with 10 mg/1 of chlorine but with sulfamic acid, continued to kill 98% or more in one hour for 12 successive increments, leaving a residual of 2.2 mg/1, then gave 96% on the 13th. Portion D, with 20 mg/1 of chlorine alone,

gave one hour 98% kill with five increments, leaving a residual of 0.20 mg/1, 95% on the next and only 63% on the seventh. Portion C was still producing 98% kill in one hour after 13 increments, with a final residual of 4.0 mg/1.

The next run was made with river water which had been autoclaved and filtered, and then had a chlorine requirement of about 2 mg/1 in 30 minutes, and approximated the quality of many country pools. Portions A and B each received 20 mg/1 of sulfamic acid and 10 of chlorine, but in A the chemicals in fairly dilute solutions had been mixed before addition to the water, while in B the chemicals were added in sequence. Portions C and D received 20 mg/1 each of sulfamic acid and chlorine, C with premixing, and in D in sequence. G, a control, received at first 10 mg/1 of chlorine alone. The increments in this run had a volume each of about 1% of the original water.

As shown in Table 2, A did not give a 98%-one hour kill even with the first application of sewage, although having a residual of 7.8 mg/1. In five hours the kill was 99.99%. With the first three increments B gave the 98-one hour kill with a residual of 4.5 mg/1, then 96% on the fourth and only 63% on the fifth. Portion C gave 98% for six increments, dropped to 93% on the 7th, 36% on the eighth, with a final residual of 8 mg/1. D was still giving 99% kill in 1 hour after eight increments, which brought the sewage content to about 10% of the original content. The control gave 99.9% kill in 15 minutes with the first increment, but no residual remained. Another 10 mg/1 of chlorine was added, and with two more additions of sewage 98% kill was given in 1 hour. The control then exhausted, gave disinfection of only about one-third as much sewage, or killed only one-third as many bacteria as Portion B.

This experiment also indicated that premixed doses of sulfamic acid and chlorine in the ratio of 2 to 1 gave poorer results than equal mixtures, and in either ratio, better kill was obtained with chemicals added in sequence. In this same experiment, a mixture of 40 sulfamic + 20 chlorine gave poor results. In other runs, amounts of acid up to 100 mg/1 even with very large doses of chlorine were not successful.

The third experiment, run with Lawrence tap water, is shown in Table 3. Each increment of sewage was about one-half

TABLE 1.—COMPARISON OF SULFAMIC ACID-CHLORINE WITH CHLORINE ALONE

	A		B		C		D	
Sulfamic acid, mg/1	20		0		20		0	
Chlorine, mg/1	10		10		20		20	
Added in sequence								
Daily Sewage Increment	%	OT	%	OT	%	OT	%	OT
1	99.99	6.5	99.99	4.0	99.99	14	99.99	13
2	99.99	6.0	99.99	1.4	99.99	12	99.99	10
3	99.9	5.0	99.99	.19	99.99	10	99.99	4.0
4	99.99	5.0	88	.00	99.99	10	99.99	1.2
5	99.9	3.5	0		99.9	5.5	99.99	.18
6	99.9	3.5			99.9	5.5	95	trace
7	99.9	3.4			99.99	5.5	63	.00
8	99.9	3.2			99.99	5.5	0	
9	99.7	2.8			99.9	5.0		
10	99.7	2.8			99.99	5.0		
11	99.4	2.5			99.9	4.6		
12	99.9	2.2			99.99	4.0		
13	96	1.7			99.2	3.8		
					could have continued			
Bacteria killed, millions								
total	577		100		813		193	
per mg of chlorine	58		10		41		10	

Water used was high in solids and alkalinity; chlorine requirement = 1.2 mg/1, 24 hr.

% = per cent of bacteria killed in 1 hour. OT = o-tolidin residual.

TABLE 2.—COMPARISON OF SULFAMIC ACID-CHLORINE WITH CHLORINE ALONE AND COMPARISON OF PRE-MIXING WITH ADDITION IN SEQUENCE

	A		B		C		D		Control	
Sulfamic acid, mg/l	20		20		20		20		0	
Chlorine, mg/l	10 I*		10 S*		20 I*		20 S*		20	
Daily Sewage Increment	%	OT	%	OT	%	OT	%	OT	%	OT
1	81	8.0	99.6	6.5	99.9	14	99.8	15	99.9	1.3
2	65	7.8	99.9	6.5	99.9	14	99.9	15	99.9	1.7
3			99.9	4.3	99.9	13	99.9	14	99.5	.75
4			96	4.0	99.9	12	99.9	12	0	
5			81	3.5	99.9	11	99.9	12		
6			0		99.9	10	99.9	10		
7					93	8	99.9	8		
8					36	8	99	8		
							could have continued			
Bacteria killed, millions										
total	65		288		478		530		187	
per mg of chlorine	7		29		24		27		9	

Filtered river water used; chlorine requirement = 2 + mg/1 in 30 minutes.

* I indicates chemicals in dilute solution mixed before adding to water.

* S indicates addition of chemicals in sequence.

% = per cent of bacteria killed in 1 hour. OT = o-tolidin residual.

of one per cent of the original volume. The control received 8 mg/1 of chlorine alone. The portion marked B received 10 mg/1 of sulfamic acid, followed by 8 of chlorine, a ratio close to the requirement for n-chloro acid. Portion D received 10 mg/1 of sulfamic acid and 16 of chlorine, or about the theoretical ratio for n-n-dichloro acid. Portion B gave 99.99% kill in one hour with 6 increments of sewage, 99.0% with the next with a residual of 4.5 mg/1, and 88% with the 8th. Portion D gave one hour kills of 99.99% with 12 increments, 99.9% on the 13th with a residual of 5.3 mg/1, and was still giving 98% on the 14th. The control gave 99.99% with three increments, with a residual of 0.20 then 96.0% on the fourth, and no further kill.

TABLE 3.—COMPARISON OF SULFAMIC ACID-CHLORINE WITH CHLORINE ALONE

	B		D		E	
Sulfamic acid, mg/1	10		10		0	
Chlorine, mg/1 Added in sequence	8		16		8	
Daily Sewage Increment	%	OT	%	OT	%	OT
1	99.99	8.0	99.99	16	99.99	8.0
2	99.99	6.3	99.99	11	99.99	3.5
3	99.99	5.3	99.99	11	99.99	0.2
4	99.99	5.0	99.99	11	96.0	0.15
5	99.99	4.8	99.99	9.5		
6	99.99	4.5	99.99	9.5		
7	99.0	4.0	99.99	9.5		
8	88.0	3.8	99.99	7.5		
9			99.99	6.8		
10			99.99	6.5		
11			99.99	5.8		
12			99.99	5.3		
13			99.99	4.5		
14			98.0	3.3		
			could have continued			
Bacteria killed, millions						
total	416		835		222	
per mg of chlorine	52		52		28	

Lawrence tap water used; very little chlorine requirement.

% = per cent of bacteria killed in 1 hour. OT = o-tolidin residual.

Table 3 also shows the number of bacteria killed by the three portions. With chlorine alone, about 28 million were killed per milligram, but with the sulfamic acid about twice as many were killed for the same dose.

4. Measurement of Active Concentrations

It was found that residuals in mixtures of sulfamic acid with chlorine or with chloramines from ammonia could be measured by the ordinary ortho-tolidin technic. For reasons not known, OTA measurements were uncertain and in fact misleading. In some of our experiments the 98% kill was delayed beyond one hour in waters which showed residuals of from 1.0 to 1.7 mg/1, while in the same waters dosed with chlorine alone the 98%-one hour kill was obtained with residuals as low as 0.2 to 0.8 mg/1. In waters which had a low original chlorine demand, and when the sewage increments were not too heavy, the mixtures with these residuals given above gave very effective kills in one to three hours added contact, but in the waters with very high organic demand, either originally or from heavy doses of sewage, occasionally even higher residuals showed long delayed kills.

It will therefore be advisable to establish the same minimum residual for each water, depending on the original organic content and any demand introduced in any way. Our results indicate that when the OT residual drops to perhaps 1.5 mg/1, and certainly to 1.0, any remaining bactericidal power should be considered as relatively slow and perhaps uncertain, and preferably ready for reinforcement. But this problem must be ascertained for any given conditions.

Other Observations

As indicated above, the typical chlorine odor was not found in proper mixtures of sulfamic acid and chlorine, even if the chlorine dose was as high as 20 mg/1, and even when the water was violently agitated. There was no indication of any chlorophenol taste in any of the waters we tested. In a few of the longer runs, an odor like that of nitrogen trichloride was observed as the active killing power approched exhaustion.

Shaking of comparable mixtures and of chlorine and chloramine solutions gave a loss of chlorine at water-air-interfaces sig-

nificantly less in mixtures than in the straight chlorine or chloramine solutions as shown in Figure 3. Droplet aeration gave similar results. A few observations on comparable solutions exposed to direct sunlight showed much lower losses in mixtures than in the other solutions.

There were definite indications that sulfamic acid-chlorine combinations were much more efficient than chloramines at pH levels of 7 or higher. This is an important feature which must be further investigated.

Although activity against algal growths has not been specifically studied, it was noted that none of the sulfamic acid-mixtures standing in the laboratory or exposed to sunlight gave any sign of growth of algae.

Summary

It appears definite that a given amount of chlorine, properly mixed with sulfamic acid will give greater kills of bacteria than the same amount of chlorine alone or as chloramine.

The active residuals of such mixtures can be measured by the usual ortho-tolidin technic but a new concept of the degree of a minimum killing residual must be established. The OTA technic does not appear to be of use.

Subject to further study, it seems preferable to add the chemicals in sequence rather than after mixing.

Loss of active chlorine due to sunlight, wind and splashing, and casual organic matter are definitely reduced.

Although originally these studies were made with swimming pool sanitation in mind, there seems to be no reason why the mixtures would not be useful wherever a controlled "stabilized" chlorine content might be employed. Full scale investigations in actual practice in pools and elsewhere will of course be needed to confirm the indications shown.

* * *

My appreciation is due to Mr. John E. Delaney, Chief of Laboratory of the Lawrence Experiment Station, and his assistants for bacterial work and valuable ideas; and to Mr. W. P. Cooke, of the Industrial and Biochemicals Department, and Dr. E. A. Savinelli, of the Engineering Department, of the E. I.

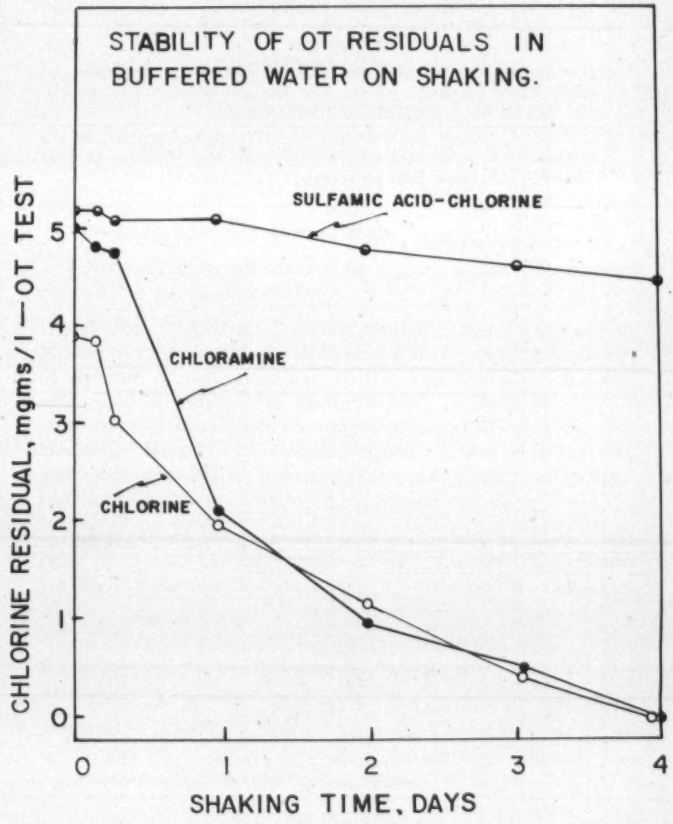
STABILITY OF OT RESIDUALS IN BUFFERED WATER ON SHAKING.
SULFAMIC ACID-CHLORINE
CHLORAMINE
CHLORINE
CHLORINE RESIDUAL, mgms/l — OT TEST
0
1
2
3
4
5
SHAKING TIME, DAYS

FIG. 3

Dupont de Nemours & Co. for the original reports which inspired this study, and further assistance and advice.

References

1. McCarthy, J. A., "Bromine and Chloride Dioxide in Treatment of Public Water Supplies," Journal New England Water Works Association, Vol. 58, No. 1, p. 46, March, 1944.
2. Butterfield, C. T., et al, "Influence of pH and Temperature on the Survival of Coliform and Enteric Pathogens when Exposed to Free Chlorine," U.S. Public Health Reports, 58, 1837 (1943).
3. Cooke, W.P.
   Savinelli, E. A.
   Private communication

PROCEEDINGS

JANUARY 1960 MEETING

HOTEL STATLER-HILTON, BOSTON, MASS.

THURSDAY, JANUARY 21, 1960

President Warren A. Gentner in the Chair.

Secretary Knox announced the election of the following to membership in the Association:

Robert C. Marini, Sanitary Engineer, Camp, Dresser & McKee, Boston, Mass.; Frank Morgenroth, Consulting Engineer, Morgenroth & Associates, Inc., Boston, Mass.; William W. Nelson, Sales Engineer, The Foxboro Co., Foxborough, Mass.; John A. Worthley, Chemist and Superintendent of Filtration, Greenwich Water Co., Greenwich, Conn.; Henry H. Reed, Jr., Superintendent, Water Department, Cohasset, Mass. *Associate:* Clavin-Fielding Co., 32 Becker Ave., Riverside, R. I.

A paper on "Disinfection of Small Swimming Pools" was read by Joseph A. McCarthy, Chief, Lawrence Experiment Station, Massachusetts Department of Public Health, Lawrence, Mass.

A movie, "Automatic Diatomaceous-Earth Filters for Public Water Supplies," a 16-mm sound film, was shown.

A paper on "Description of Installation of Automatic Diatomaceous-Earth Filters at Saratoga, California," was read by George A. Pettersen, New England District Manager, De-Laval Steam Turbine Co., Boston, Mass.

FEBRUARY 1960 MEETING

HOTEL STATLER-HILTON, BOSTON, MASS.

THURSDAY, FEBRUARY 18, 1960

President Warren A. Gentner, Presiding

Secretary Knox announced the election of the following new members:

William Ammann, Superintendent, Water Department, Palmer, Mass.; Philip J. Barbato, Project Engineer, Edwards & Kelcey, 470 Atlantic Ave., Boston, Mass.; John J. Carroll, Superintendent, Public Works, Lexington, Mass.; Richard D. Chinnock, Water Commissioner, North Conway, N. H.; Donald E. Cullivan, Sanitary Engineer, Camp, Dresser & McKee, Boston, Mass.; Cormello Diorio, Superintendent, Water Department, Methuen, Mass.; Charles H. Flavin, Jr., Senior Civil Engineer, Edwards & Kelcey, Boston, Mass.; John O. Grace, Vice-Chairman, Board of Water Commissioners, Holyoke, Mass.; John M. Kinsey, Commissioner of Public Works, Fitchburg, Mass.; Robert A. McCracken, Senior Sanitary Engineer, Water Supply Section, Massachusetts Department of Public Health, Boston, Mass.; John W. Meany, Acting Superintendent, Water Department, Gardner, Mass.; John K. Mitchell, Registrar, Water Department, Gardner, Mass.; Henry P. Tilden, Jr., Superintendent, Water Department, Weymouth, Mass.; Harvey S. Westover, Superintendent, Shirley Water District, Shirley, Mass.

Associate Member:

Prufcoat Laboratories, Inc., 63 Main St., Cambridge, Mass.

PRESIDENT GENTNER. It is with deep regret that I announce the passing of one of our most highly esteemed members —Caleb Mills Saville.

Mr. Saville joined the New England Water Works Association in 1893, served as president in 1917, and was elected to honorary membership in 1929.

It was my privilege to have been associated with Mr. Saville for the greater part of my professional career, and indeed

it was a privilege. He was a very capable, far-seeing engineer—a man with vision—and it was through his efforts that the water-supply system of the city of Hartford grew to what it is today.

May we all stand for one moment in silence as a tribute.

[*All the members stand in silence.*]

A talk on "Why You Should Do Something about Your Tensions" was given by Dr. Rosser P. Atkinson, member of staff, Lahey Clinic and New England Baptist Hospital, Boston, Mass.

A paper on "Cleaning Debris from Reservoir while Maintaining Service," prepared by Peter C. Karalekas, Chief Engineer, Municipal Water Works, Springfield, Mass., and Charles A. Manganaro, Bogert & Childs, New York, N. Y., was read by Mr. Karalekas.

March 1960 Meeting

Hotel Statler-Hilton, Boston, Mass.

Thursday, March 17, 1960

President Warren A. Gentner, Presiding

Secretary Knox announced the election of the following to membership in the Association:

Floyd E. Brown, Civil Engineer, Cleverdon, Varney & Pike, Boston, Mass.; Howard G. Egginson, Sales Representative, Precision Chemical Pump Corp., Framingham, Mass.; Edward H. Fletcher, Acting Superintendent, Water Department, Shrewsbury, Mass.; Allan Grieve, Jr., Principal Civil Engineer, Metropolitan District Commission, Water Department, Boston, Mass.; Wesley R. Hazelton, Superintendent, Water Department, Hartford, Vt.; Warren H. Oster, Consulting Engineer, Hayden, Harding & Buchanan, Inc., Boston, Mass.; Stuart C. Peterson, Consulting Engineer, Hayden, Harding & Buchanan, Inc., Boston, Mass.; John M. W. Sargent, Senior Civil Engineer, Metropolitan District Commission, Water Division, Boston, Mass.; George E. Sherman, Principal Civil Engineer, Metropolitan District Com-

mission, Water Division, Boston, Mass.; Warren M. Walkup, Jr., Superintendent, Water Department, Ashland, Mass.; J. Andre Provencial, Temporary Superintendent, Water Department, Fitchburg, Mass.

Associate Member:

Rotary Valve & Engineering Corp., Birmingham, Ala.

PRESIDENT GENTNER. In accordance with Article VII of the Constitution, the Association is directed to elect a nominating committee at this meeting, to present a list of nominations of officers to be elected for the ensuing year. The Chair will entertain a recommendation.

RICHARD H. ELLIS. As has been the custom for several years past, I move that the last five living past presidents, exclusive of those now serving on the Executive Committee—namely, Messrs. Stanley M. Dore, William B. Duffy, Roger G. Oakman, Alan F. McAlary and Edward L. Tracy, with Mr. Oakman as chairman—be elected to serve as the nominating committee for this year.

The motion was duly seconded; the question was put, and the motion was declared carried.

A talk on "To Market, to Market, to Buy a Fat Pig" was given by Charles M. Healey, Jr., Purchasing Agent, City of Springfield, Mass.

A paper on "Design and Construction of Water Main Crossing under Taunton River, Somerset, Mass.," was read by George A. Howland, Project Engineer, Whitman & Howard, Inc., Boston, Mass.

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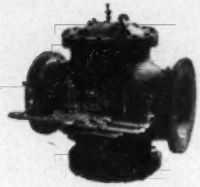

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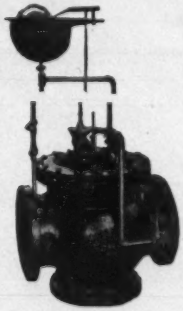

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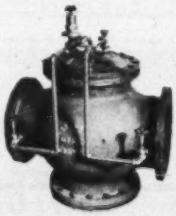

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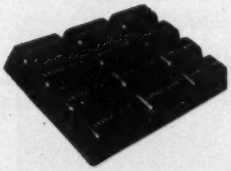

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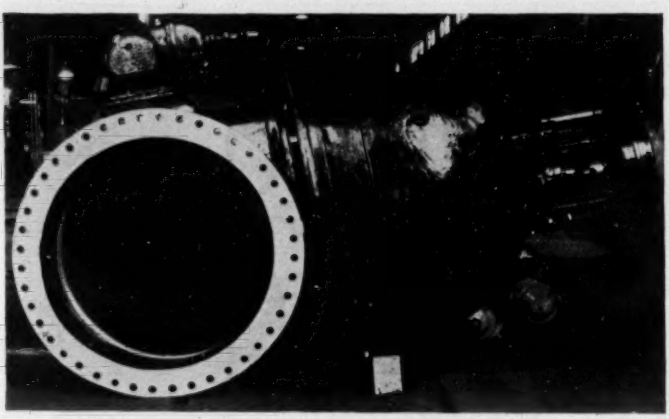

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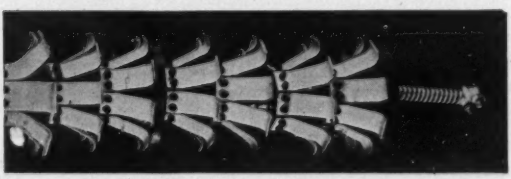

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